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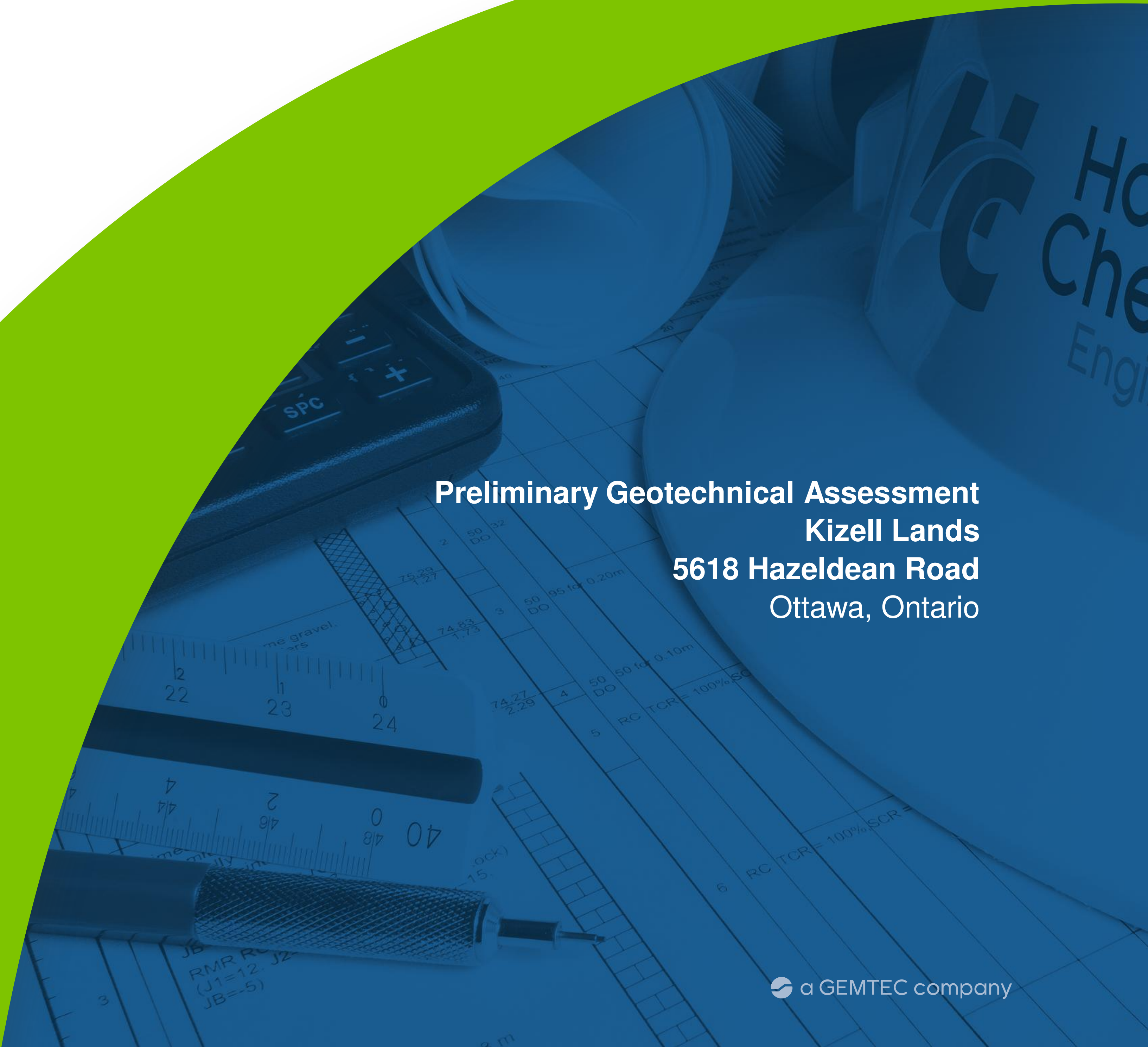

Preliminary Geotechnical Assessment
Kizell Lands
5618 Hazeldean Road
Ottawa, Ontario

a GEMTEC company

Houle Chevrier Engineering

Submitted to:

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1.0 INTRODUCTION

This report provides a preliminary geotechnical assessment for the Kizell Lands in Ottawa, Ontario. The purpose of the report is to provide preliminary geotechnical guidelines and recommendations for the proposed development based on available test pit and borehole information at and in the vicinity of the site.

We carried out the following tasks as part of our scope of work:

- A site visit was carried out on December 3, 2015 to view the surface conditions.
- Air photographs for the period from 1932 to 2014 were reviewed.
- Available test pit and borehole information was collected and compiled from our files.

Preliminary geotechnical guidelines and recommendations for the proposed development were then prepared for planning purposes based on the available, widely spaced test hole information. This preliminary report should not be used for design and construction at any specific location, nor is the report to be used as a replacement for site specific geotechnical investigations. Furthermore, this report does not provide guidelines and recommendations for the proposed north-south aligned Stittsville trunk sewer, nor does it provide information for the design of a grade separated crossing of the proposed arterial roadway at Hazeldean Road.

2.0 PROJECT DESCRIPTION

The subject property has an approximate area of 89 hectares (220 acres) and is located at 5618 Hazeldean Road in Ottawa, Ontario (refer to Key Plan, Figure 1).

Plans are being prepared to develop the property for residential, commercial and institutional uses. The development will include: houses, high density residential housing, an elementary school, mixed-use commercial development, neighbourhood commercial lands, a district park, neighbourhood parks, and a stormwater management pond. Access to the development will be via an internal roadway system, including a north-south aligned arterial roadway. Site services will include watermains, and storm and sanitary sewers. A copy of the concept plan for the site is provided in Appendix A.

The subject property has generally been used for agricultural purposes since its patent in 1824. The majority of the site currently consists of agricultural lands. The lands are relatively flat, and slope gently downward to the north and east from a high point located at about the midpoint of the west side of the site (i.e., from the bedrock outcrop in Area 1, as shown on Figure 2). The site is covered mostly with crops, except for a treed area which is located at the aforementioned high point and southeast of the high point.

3.0 SURFICIAL AND BEDROCK GEOLOGY MAPS

Based on surficial geology maps of the Ottawa area, it is expected that the overburden at the site is characterized primarily by deposits of silty clay of marine origin over glacial till. There appears to be a localized area in the west part of the site that is characterized by exposed bedrock and/or bedrock at shallow depth.

The overburden thickness is indicated to range from 0 to 3 metres within the southwest part of the site, increasing to between 5 and 10 metres to the north and east.

The bedrock is mapped as interbedded silty dolostone, lithographic to fine crystalline limestone, oolitic limestone, shale and fine grained calcareous quartz sandstone of the Gull River formation. There are no bedrock faults mapped at the site.

4.0 PREVIOUS INVESTIGATIONS BY HOULE CHEVRIER ENGINEERING LTD. AND OTHERS

Previous investigations have been carried out in the study area by Houle Chevrier Engineering Ltd. (HCEL) and other geotechnical firms. The results of these studies are provided in the following reports:

- “Preliminary Geotechnical Investigation, Fernbank Community Design Plan, Ottawa, Ontario”, dated May 28, 2007, prepared by Houle Chevrier Engineering Ltd. (report number 06-384).
- “Geotechnical Investigation, Proposed Arterial Road and Abbott Street Extension, Fernbank Development, Ottawa, Ontario”, dated September 2013, prepared by Houle Chevrier Engineering Ltd. (report number 12-247).
- Borehole logs from a geotechnical investigation for a sewer outlet prepared by Golder Associates Ltd. (report number 001-2225).
- “Final Pavement Report for Proposed Widening of Hazeldean Road, Iber Road to Kincardine Drive, Ottawa, Ontario”, dated March 2005, prepared by Jacques Whitford Ltd. (report number ONO11725).

The results of the previous geotechnical investigations are provided in Attachments B to E following the text of this report. It should be noted that the borehole and test pit information does not reflect changes due to subsequent construction.

5.0 SUBSURFACE CONDITIONS

The following presents an overview of the subsurface conditions encountered in the test pits and boreholes advanced during the previous investigations. For the purposes of this study, we have broadly divided the site into three (3) main areas, numbered Areas 1 to 3. The generalized horizontal extent of these areas is provided on the Site Plan, Figure 2.

5.1 Area 1

Based on the site reconnaissance together with surficial geology maps, there appears to be an area along the southwest part of the site that is characterized primarily by exposed bedrock or bedrock at shallow depth.

An area of exposed limestone bedrock with open vertical joints and juvenile Karstic features was observed within the west part of the site during the site reconnaissance (refer to Site Plan, Figure 2)

One test pit (test pit 10) encountered thin deposits of weathered silty clay and silt followed by glacial till. The groundwater level in the open test pit was at about 1.5 metres below ground surface at the time of the investigation (November 2006).

5.2 Area 2

The test pits and boreholes that were advanced in Area 2 encountered a surficial layer of topsoil about 0.1 to 0.7 metres thick, followed by silty clay and clayey silt. The upper 0.7 to 3.5 metres of the silty clay/clayey silt is weathered to a grey brown crust. Below the zone of weathering, the silty clay/clayey silt is grey and has a firm to stiff consistency. In situ vane shear strength tests carried out in the grey silty clay in boreholes 2 and 3 gave undrained shear strength values ranging from 44 to 78 kilopascals; the measured undrained shear strength in the bucket samples recovered from test pits 5, 9 and 18 ranged from 28 to 90 kilopascals. It should be noted that the vane shear strengths in the bucket samples may not be representative, since there would likely have been some disturbance and softening of the silty clay during excavation. Glacial till was encountered below the silty clay in test pit 11 and borehole 00-3 at depths of about 3.3 and 3.8 metres below ground surface.

The following observations were made with respect to the groundwater levels:

- The groundwater levels measured in standpipes sealed in the boreholes 2 and 3 ranged from 1.4 to 1.5 metres below ground surface on March 7, 2007. The groundwater level in borehole 00-5 was at 2.2 metres below ground surface on November 10, 2000.
- Substantial groundwater inflow was observed from fissures and/or tile drains in the silty clay in test pits 1, 3, 9. At the locations of test pits 3 and 9, the vertical sides of the test pits collapsed during excavating.
- Groundwater inflow was observed in all of the test pits in Area 2. The depth to groundwater inflow ranged from ground surface (test pit 9) to 2.5 metres below ground surface (test pit 11) in November and December 2006. It should be noted that these groundwater levels do not represent stabilized groundwater conditions.
- The groundwater levels measured in standpipes installed in test pits 2, 9 and 18 ranged from ground surface to 1.6 metres below ground surface on January 3, 2007. The

groundwater levels measured in the standpipes were at or above those that were measured following excavation.

- Some of the agricultural fields in Area 2 are tile drained. The tile drains could affect the seasonal shallow groundwater levels on the site. Furthermore, substantial groundwater inflow should be expected from the tile drains, particularly during wet periods of the year.

5.3 Area 3

The test pits and boreholes that were advanced in Area 3 encountered a surficial layer of topsoil about 0.3 metres thick, followed by silt, sandy silt, silty sand and silty clay. The upper 2.4 to 2.9 metres of the silty clay encountered in the previous test pits and boreholes is weathered to a grey brown crust. Below the zone of weathering, the silty clay is grey and has a soft to stiff consistency. In situ vane shear strength tests carried out in the grey silty clay in borehole 1 gave undrained shear strength values ranging from 25 to 68 kilopascals; the measured undrained shear strength in the bucket samples recovered from test pits 2, 6, and 19 ranged from 19 to 40 kilopascals. It should be noted that the vane shear strengths in the bucket samples may not be representative, since there would likely have been some disturbance and softening of the silty clay during excavation. In situ vane shear strength testing carried out in the grey silty clay encountered in the previous boreholes advanced by Jacques Whitford Ltd. along Hazeldean Road gave undrained shear strength values of 25 to over 100 kilopascals.

The following observations were made with respect to the groundwater levels:

- The groundwater levels measured in the standpipe sealed in borehole 1 was at 1.9 metres below ground surface on March 7, 2007.
- Substantial groundwater inflow was observed from fissures and/or tile drains in the silty clay in test pit 2.
- Groundwater inflow was observed in all of the test pits in Area 3. The depth to groundwater inflow ranged from 1.7 to 2.2 metres below ground surface on November 15, 2006. It should be noted that these groundwater levels do not represent stabilized groundwater conditions.
- The groundwater levels measured in standpipes installed in test pits 2 and 6 were at 0.2 and 0.8 metres below ground surface on January 3, 2007. The groundwater levels measured in the standpipes were at or above those that were measured following excavation.
- Some of the agricultural fields in Area 3 known to be tile drained. The tile drains could affect the seasonal shallow groundwater levels on the site. Furthermore, substantial groundwater inflow should be expected from the tile drains, particularly during wet periods of the year.

6.0 PRELIMINARY GEOTECHNICAL GUIDELINES AND RECOMMENDATIONS

6.1 General

This section of the report provides preliminary engineering guidelines and recommendations on the geotechnical design aspects of the project based on widely spaced boreholes and test pits from previous investigations by HCEL and others. Detailed site specific investigations are recommended during design. Site specific investigations may reveal soil and groundwater conditions that were not identified in this study due to natural variability of ground conditions, which may affect design requirements.

The information in this report is based on our interpretation of the available test pit information, and the project requirements. It is stressed that the information in the following sections is provided for the guidance of the designers and is intended for this project only. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The professional services retained for this study include only the geotechnical aspects of the subsurface conditions at this site. The results of a Phase I Environmental Site Assessment of the property are provided in a separate report by HCEL.

6.2 Excavation

6.2.1 Overburden Excavation

The excavations for the proposed buildings and services may be taken through fill, topsoil, silty clay/clayey silt, silt, sandy silt, silty sand and glacial till. The sides of the excavations should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. According to the Act, the shallow native overburden deposits can be classified as Type 3 and, accordingly, allowance should be made for excavation side slopes of 1 horizontal to 1 vertical extending upwards from the base of the excavation. As an alternative to the sloping the excavations, the site service installations could be carried out within a tightly fitting, braced steel trench box, which is specifically designed for this purpose.

Excavation of the native soils above the groundwater should not present significant excavation constraints. In contrast, excavation in the native silt, sandy silt and silty sand below the groundwater level could present constraints. Groundwater inflow from silt, sandy silt, and silty sand deposits, where encountered, could cause sloughing of the sides of the excavation and disturbance to the soils at the bottom of the excavation. Flatter side slopes and or drainage measures may be required if excavation is required below the groundwater level in silt, sandy silt and silty sand deposits. It is our experience that excavation for site service installations to shallow depth within these silty or sandy deposits can usually be carried out within a braced steel trench box specifically designed for this purpose, in combination, where necessary, with steel plates advanced along the sides of the trench box to below the level of excavation. In this case, the groundwater inflow should be controlled throughout the excavation and pipe laying operations by pumping from sumps within the excavation. Notwithstanding, some disturbance and loosening of the subgrade materials could occur, and allowance should be made for subexcavation and additional pipe bedding (sub-bedding) material, as discussed later in this report.

Based on our observations on site, groundwater inflow from the overburden deposits into the excavations should be controlled by pumping from filtered sumps within the excavations. It is not expected that short term pumping during excavation will have any significant affect on nearby structures and services.

It is noted that sloughing occurred in the weathered silty clay during excavation of some of the test pits, likely due to the presence of fissures in the weathered silty clay combined with high groundwater conditions and/or significant groundwater inflow. In areas where sloughing is encountered, the excavation for the site services should be carried out within a tightly fitting, braced steel trench box.

6.2.2 Bedrock Excavation

Localized removal of competent bedrock at this site could be carried out using (a) drill and blasting, (b) hoe ramming techniques in conjunction with line drilling on close centres or (c) a combination of both. Provided that good bedrock excavation techniques are used, the competent bedrock could be excavated using vertical side walls.

Any blasting should be carried out under the supervision of a blasting specialist engineer. As a guideline for blasting, the suggested peak vibration limits at the nearest structure or service are provided in Table 6.1.

Table 6.1 – Peak Vibration Limits

Frequency of Vibration (Hz)	Vibration Limits (millimetres/second)
<10	5
10 to 40	5 to 50 (interpolated)
>40	50

It is pointed out that these criteria, although conservative, were established to prevent damage to existing buildings and services in good condition; more stringent criteria may be required to prevent damage to freshly placed (uncured) concrete or vibration sensitive equipment or utilities. Monitoring of the blasting should be carried out to ensure that the blasting meets the limiting vibration criteria. Pre-construction condition surveys of nearby structures and existing buried services are considered essential. The effects due to vibration from blasting can be controlled by limiting the size and amount of charge, using delayed detonation techniques, and the like. To reduce the effects of vibration on nearby services, we suggest that the separation distance between any blasting and existing underground services be at least 3 metres. Any bedrock removal within these limits could be carried out using hoe ramming techniques in conjunction with line drilling on close centres. It is noted that the cost of bedrock removal generally increases the closer the bedrock removal is to any existing structures or services.

As an alternative to blasting, bedrock removal could be carried out using large hydraulic excavation equipment in combination with hoe ramming. Line drilling on close centres could be used to reduce, not prevent, over break and under break of the bedrock excavation and to define the limit of excavation next to existing structures and services. For the bedrock at this site, it is suggested that allowance be made for line drilling 75 to 100 millimetre diameter holes on 200 to 300 millimetre centres. The vibration effects of hoe ramming are usually minor and localized. Monitoring of the hoe ramming could be carried out, at least initially, to measure the vibrations to ensure that they are below the acceptable threshold value.

Provided that good bedrock excavation techniques are used, the bedrock could be excavated using vertical side walls. Any loose rock should be scaled from the side of the excavation.

The bedrock at this site has near horizontal bedding planes and near vertical inclined joints, some of which are open to ground surface. Therefore, some vertical and horizontal over break of the bedrock should be expected. The exposed bedrock within the west part of the site has open, vertical joints; after blasting some large pieces of bedrock may require mechanical breaking to allow handling and disposal. Vertical over break will naturally occur along the bedding planes; as such, additional granular bedding material should be expected for the site services and additional granular fill/concrete should be expected for the house/structure foundations.

6.2.3 Groundwater Pumping

Except as noted above for the silt, sandy silt, silty sand deposits, groundwater inflow from the overburden deposits should be controlled by pumping from within the excavations. Significant groundwater inflow was observed from fissures in the weathered silty clay in some of the test pits that were advanced during a previous investigation by HCEL. Allowance should be made for significant pumping where these conditions and/or if existing agricultural tile drains are encountered.

Groundwater inflow from the bedrock into the excavations for the site services should be expected and should be handled by pumping from within the excavations. Significant groundwater pumping should be anticipated and basal heaving of the soil may occur in the bottom of service trenches in areas where highly permeable bedrock exist at shallow depth below the bottom of the excavation. Furthermore, significant groundwater inflow may occur from open joints in the bedrock, such as those observed in the exposed bedrock within the west part of the site. As a consequence, it may be necessary to remove and replace the native overburden below the bottom of the excavation with compacted granular material.

A Permit to Take Water (PTTW) will be required for pumping from within the excavations, in accordance with Ministry of the Environment and Climate Change (MOECC) requirements. The type of permit (i.e., Category 2 or 3) will depend on the depth of the excavations relative to the groundwater level, type of soil within the anticipated depth of excavation, and project duration. A Category 3 permit will be required if the project is to be staged over several years. Issuance of the permit by the MOECC usually takes about 90 business days.

6.3 Grade Raise Fill Restrictions

6.3.1 Area 1

Area 1 is generally characterized by exposed bedrock or shallow bedrock. From a geotechnical point of view, no grade raise restrictions are anticipated in Area 1.

6.3.2 Area 2

Area 2 is underlain by deposits of sensitive silty clay/clayey silt, which have a firm to stiff consistency. The placement of fill material in Area 2 must be controlled so that the stress imposed by the fill material does not result in excessive consolidation of the grey silty clay deposits. The settlement response of the silty clay deposits to the increase in stress caused by fill material is influenced by variables such as the existing effective overburden pressure, the past preconsolidation pressure of the silty clay, the compressibility characteristics of the silty clay, and the presence or absence of drainage paths, etc. It is well established that the settlement response of silty clay deposits can be significant when the stress increase is near or above the preconsolidation pressure. For preliminary design purposes, the grade raise fill restriction could be about 1.5 to 2.0 metres, assuming that conventional earth and granular fill is used in and around the proposed structures. The site service and roadway design should take this restriction into account.

6.3.3 Area 3

Area 3 is underlain by deposits of silty clay, which appear to have a soft to stiff consistency. The shear strength of the silty clay is generally greater than 25 kilopascals, except for some possible localized, softer zones. The silty clay in Area 3 has a lower capacity to support loads imposed by grade raise fill material than that in Area 2. The placement of fill material in Area 3 must therefore be carefully controlled so that the stress imposed by the fill material does not result in excessive consolidation of the grey silty clay deposits. For preliminary design purposes, the grade raise fill restriction could range from about 1.2 to 1.5 metres; the amount of grade raise filling could be somewhat less than 1.2 metres where the silty clay has a soft consistency. As indicated above, the site service and roadway design should take this restriction into account.

6.3.4 Alternative Fill Types

The above grade raise restrictions are based on the use of the native soils (such as silty clay, clayey silt, sandy silt and silty sand) as grade raise fill material. Other materials could be considered to increase the thickness of grade raise fill in Areas 2 and 3, such as:

- Relatively lightweight fill material, such as polystyrene insulation,
- Isofill manufactured by Lafarge. or
- Clear crushed stone.

Further information on the use of light weight fill could be provided upon request.

6.4 Proposed Single Family Homes and Townhouses

6.4.1 Subgrade Preparation and Engineered Fill

The excavations for the houses should be taken through any surficial organic deposits and fill. The native overburden deposits are considered suitable for the support of structures on spread footing foundations. Silt, sandy silt and silty sand deposits below the groundwater level, where encountered, may become disturbed during excavation and, thereby, may not provide suitable support. In these areas, pre-drainage of the overburden deposits may be required; this could be achieved through the installation of the site services and, if necessary, by ditching in advance of excavation.

It is our experience that the upper part of the weathered silty clay (i.e., within 0.3 to 0.5 metres from original ground surface) may be impacted by past frost action. During removal of the topsoil and fill material, the upper part of the silty clay could unavoidably peel upwards and become disturbed. Where this occurs within the proposed houses, the disturbed silty clay should be removed and replaced with compacted granular material.

In areas where proposed the founding level is above the level of the native soil or bedrock, or where subexcavation of disturbed material is required below proposed founding level, imported granular material (engineered fill) should be used. The engineered fill should consist of granular material meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular B Type II and should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density. In areas where groundwater inflow is encountered, pumping should be carried out from sumps in the excavation during placement of the engineered fill. In areas where silt, sandy silt or silty sand deposits exist below the engineered fill, it may be necessary to place a relatively thick lift of engineered fill on the silt, sandy silt or silty sand and to compact it statically (without vibration) with a steel drum roller to avoid disturbance of the subgrade. To allow spread of load beneath the footings, the engineered fill should extend horizontally at least 0.2 metres beyond the footings and then down and out from the edges of the footings at 1 horizontal to 1 vertical, or flatter. The excavations for the residential dwellings should be sized to accommodate this fill placement. Currently, OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular A materials. Since the source of recycled material cannot be determined, it is suggested that for environmental reasons any granular materials used below founding level be composed of virgin material only.

6.4.2 Foundation Design

6.4.2.1 Area 1

Footings bearing on the native, undisturbed overburden deposits or engineered fill should be sized using an allowable bearing pressure of 100 to 150 kilopascals. The settlement of the footings should be less than 25 millimetres, provided that any loose or disturbed soil is removed from the bearing surfaces.

An allowable bearing pressure of 500 kilopascals could be used for footings bearing on or within bedrock. In this case, the settlement of the footings should be negligible, provided that the bearing surfaces are cleaned of soil. It is our experience that some unavoidable overblasted bedrock could occur below the house foundations. Up to about 1 metre of overblasted bedrock could be left in place below the footings, provided that it is well shattered and graded and heavily compacted with a large (10 tonne minimum) steel drum roller prior to placing the footings.

There may be areas on this site where the subgrade material at founding level transitions from overburden to bedrock. To reduce the potential for cracking of basement foundation walls above abrupt transitions from overburden to bedrock, it is suggested that the foundations walls in the transition zone be suitably reinforced.

6.4.2.2 Area 2

The allowable bearing pressure used to size the footings will depend on the depth of the grey silty clay below the footings, the shear strength and consolidation characteristics of the silty clay and the amount of grade raise fill placed around the house and in the garage, all of which are not known at this time. Given that firm to stiff silty clay is expected, the preliminary allowable bearing pressures used to size the footings for houses should be about 100 kilopascals.

6.4.2.3 Area 3

The allowable bearing pressure used to size the footings will depend on the depth of the grey silty clay below the footings, the shear strength and consolidation characteristics of the silty clay and the amount of grade raise fill placed around the house and in the garage, all of which are not known at this time. Given that soft to firm silty clay is expected, the allowable bearing pressures used to size the footings for houses should be in the range of 75 to 100 kilopascals.

6.4.3 Frost Protection of Footings for Houses

All exterior footings should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated, unheated exterior footings adjacent to surfaces which are cleaned of snow cover during the winter months should be provided with a minimum of 1.8 metres of earth cover. Alternatively, the required frost protection could be provided by means of a combination of earth cover and extruded polystyrene insulation. Further details regarding the insulation of foundations could be provided at the detailed design stage, if necessary.

6.4.4 Basement Foundation Wall Backfill and Drainage

In accordance with the Ontario Building Code, the following alternatives could be considered for drainage of the basement foundation walls:

- Damp proof the exterior of the foundation walls and backfill the walls with free draining, non-frost susceptible sand or sand and gravel such as that meeting OPSS requirements for Granular B Type I or II. OR
- Damp proof the exterior of the foundation walls and install an approved proprietary drainage material on the exterior of the foundation walls and backfill the walls with native material or imported soil.

A perforated plastic foundation drain with a surround of clear crushed stone should be installed on the exterior of the foundation walls. A nonwoven geotextile should be placed between the top of the clear stone and any sandy foundation wall backfill material to avoid loss of sand backfill into the voids in the clear stone (and possible post construction settlement of the ground around the houses). The top of the drain should be located below the bottom of the floor slab. The drain should outlet to a sump from which the water is pumped or should drain by gravity to a storm sewer.

6.4.5 Garage Foundation and Pier Backfill

To avoid adfreeze and possible jacking (heaving) of the foundation walls due to adfreeze between the unheated garage foundation walls and the wall backfill, the interior and exterior of the garage foundation walls should be backfilled with free draining, non-frost susceptible sand or sand and gravel such as that meeting OPSS requirements for Granular B Type I or II. The backfill within the garage should be compacted in maximum 300 millimetres thick lifts to at least 95 percent of the standard Proctor dry density value using suitable vibratory compaction equipment.

Alternatively, the interior of the garages could be filled with 19 millimetre clear crushed stone. In areas where the subgrade consists of silt, sandy silt, silty sand, or sand, a suitable nonwoven geotextile should be placed over the subgrade prior to the placement of clear stone to prevent ingress of fines into voids in the clear stone and possible settlement/cracking of the slab. Compaction of the clear stone is not considered essential.

The backfill against isolated (unheated) walls or piers should consist of free draining, non-frost susceptible material, such as sand/sand and gravel meeting OPSS Granular B Type I or II requirements. The backfill should be compacted in maximum 300 millimetres thick lifts to at least 95 percent of the standard Proctor dry density value using suitable vibratory compaction equipment.

Other measures to prevent frost jacking of these foundation elements could be provided, if required.

6.4.6 Basement Concrete Slab Support

To provide predictable settlement performance of the basement slab, all topsoil, fill material, disturbed soil, and other deleterious materials should be removed from the slab area.

The base for the floor slab should consist of 19 millimetre clear crushed stone. Allowance should be made for between 150 and 200 millimetres of base material. Compaction of the clear stone is not considered essential.

In areas where the subgrade consists of silt, sandy silt, or silty sand, a suitable nonwoven geotextile should be placed over the subgrade prior to the placement of clear stone to prevent ingress of fines into voids in the clear stone and possible settlement/cracking of the slab.

If clear crushed stone is used below the floor slab, underfloor drains are not considered essential provided that drains are installed to link any hydraulically isolated areas in the basement. The drains should outlet by gravity to a sump from which the water is pumped or drained by gravity to a sewer.

Basement floor slabs should be constructed in accordance with guidelines provided in ACI 302.1R-04 “Guide for Concrete Floor and Slab Construction”.

A polyethylene vapour barrier should be installed below the basement floor slabs.

6.4.7 Seismic Site Classification

According to Table 4.1.8.4.A of the Ontario Building Code, 2006, Site Class C should be used for the seismic design of the structures bearing on bedrock, engineered fill material over bedrock, and thin deposits of glacial till over bedrock.

Site Class D should be used for preliminary design of houses which are founded on the native deposits of silty clay, clayey silt, silt, sandy silt, silty sand, or glacial till. Site Class C may also be appropriate, depending on the depth of the bedrock.

In our opinion the soils encountered in the previous test pits and boreholes are not considered to be liquefiable or collapsible under seismic loads.

6.4.8 Effects of Agricultural Tile Drains on House Foundations

Agricultural tile drains were encountered in some of the previous test pits. Any tile drains encountered within the house excavations could be a source of significant volumes of water, which could impact on the basements of the houses. It is suggested that any drainage tiles that are within a horizontal distance of about 2 metres to the dwellings be removed and the excavation for the tiles backfilled with compacted silty clay to prevent any water flow through the tiles or trench. The silty clay could be compacted with the bucket of the excavator. Any drainage tiles that are below proposed the footings should be removed. The ends of the drains should be severed at least 2 metres outside of the proposed basement foundations to reduce the potential for post construction groundwater inflow into the basements. The excavation for the tiles should be backfilled with compacted silty clay as described above.

6.5 Commercial and Institutional Buildings

6.5.1 Subgrade Preparation and Engineered Fill

The excavations for commercial/institutional buildings should be taken through any surficial organic deposits and fill. Silt, silty sand and sand deposits below the groundwater level, where encountered, may become disturbed during excavation and, thereby, may not provide suitable support. In these areas, pre-drainage of the overburden deposits may likely be required; this could be achieved through the installation of the site services and, if necessary, by ditching in advance of excavation.

In areas where proposed founding level is above the level of the native soil or bedrock, or where subexcavation of disturbed material is required below proposed founding level, imported granular material (engineered fill) should be used. The engineered fill should consist of granular material meeting OPSS requirements for Granular B Type II and should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density. In areas where groundwater inflow is encountered, pumping should be carried out from sumps in the excavation during placement of the engineered fill. In areas where silt, sandy silt or silty sand deposits exist below the engineered fill, it may be necessary to place a relatively thick lift of engineered fill on the silt, sandy silt or silty sand and to compact it statically (without vibration) with a steel drum roller to avoid disturbance of the subgrade. To allow spread of load beneath the footings, the engineered fill should extend horizontally at least 0.2 metres beyond the footings and then down and out from the edges of the footings at 1 horizontal to 1 vertical, or flatter. The excavations for the foundations should be sized to accommodate this fill placement. Currently, OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular A material. Since the source of recycled material cannot be determined, it is suggested that for environmental reasons any granular materials used below founding level be composed of virgin material only.

6.5.2 Foundation Design

6.5.2.1 Spread Footing Design

Commercial/institutional buildings are currently planned in Areas 2 and 3. Based on the results of the available test pit and boreholes information, lightly loaded 1 and 2 storey commercial/institutional buildings of slab on grade design could likely be founded on spread or pad footings bearing on or within undisturbed silty clay. All organic material, fill material, topsoil, and loose or water softened soils should be removed within the building areas.

The bearing pressures for spread or pad footing foundations within Areas 2 and 3 at this site are based on the necessity to limit the stress increase on the softer, grey silty clay layer below the weathered crust to an acceptable level so that foundation settlements will not be excessive. Four important parameters in calculating the stress increase on the silty clay are:

- The underside of footing elevation (depth of excavation);
- The size and type (i.e., pad or strip), and loading of the foundation;
- The amount of surcharge (fill, etc.) in the vicinity of the foundation; and
- The amount of post-development groundwater lowering at the site.

The following preliminary bearing pressures could be used to assess foundation requirements:

Table 6.2 – Preliminary Bearing Pressures for Foundations

Location	Subgrade Material	Geotechnical Reaction at Serviceability Limit State, SLS (kPa)	Factored Geotechnical Resistance at Ultimate Limit State, ULS (kilopascals)
Area 2	Weathered Silty Clay	100 to 150	200 to 300
Area 3	Weathered Silty Clay	75 to 150	200 to 300

Note: The above bearing pressures assume that less than 1.0 and 0.5 metres of fill material is placed in and around the proposed buildings in Areas 2 and 3, respectively.

Some of the native soils at this site are sensitive to construction operations, from ponded water and frost action. The construction operations should, therefore, be carried out in a manner that minimizes disturbance of the subgrade surfaces.

It is our experience that the upper part of the weathered silty clay (i.e., within 0.3 to 0.5 metres from original ground surface) may be impacted by past frost action. During removal of the topsoil and fill material, the upper part of the silty clay could unavoidably peel upwards and become disturbed. Where this occurs within the proposed buildings, the disturbed silty clay should be removed and replaced with compacted granular material.

6.5.2.2 Deep Foundation Design

Based on the results of the available test pit and boreholes information, heavily loaded 1 and 2 storey buildings and multi-storey buildings could be founded on deep foundations, such as driven end bearing piles. Deep foundations may also be required for lightly loaded 1 and 2 storey buildings if the grade raise is close to the limits provided in Section 6.3.

It is common practice in the Ottawa area to use pipeline steel for piling. We suggest that a similar approach be taken for this project and that closed ended, concrete filled, steel pipe piles be used. The following pile capacities could be used for preliminary design:

Table 6.3 – Pile Type

Pile Type	Geotechnical Reaction at Serviceability Limit States (kilonewtons)	Factored Geotechnical Resistance at ULS (kilonewtons)
244 mm diameter by 12 mm thick	1,100	1,350

Note: The SLS and ULS loads assume that the yield strength of the steel is at least 340 MPa and that the piles are filled with concrete having a compressive strength of at least 30 MPa.

The downdrag loads due to consolidation of the grey silty clay should be considered in the pile design. The amount of downdrag loading which may occur will depend on the following factors:

- The pile type and size;
- The increase in stress due to grade raise filling and groundwater drawdown.

Pipe piles should be driven closed ended and fitted with 20 millimetre (minimum) thick end plates.

The bedrock in the Ottawa is typically overlain by glacial till. The glacial till in the Ottawa area is known to contain cobbles and boulders. It is possible that some of the piles may encounter refusal to driving on or within bouldery glacial till. The use of a pile with a thick wall (12 millimetres, or greater) may allow penetration of the glacial till with less damage. Notwithstanding, some problems with misalignment, plumbness, bending and/or sweeping of the piles, and hard driving conditions could occur due to the presence of cobbles and boulders above the bedrock surface. As such, allowance should be made to drive additional piles and to enlarge some of the pile caps, etc., as required. The requirement for this, if any, would have to be evaluated at the time of construction.

6.5.3 Frost Protection of Foundations

All exterior footings should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated (unheated) piers that are located in areas that are to be cleared of snow should be provided with at least 1.8 metres of earth cover for frost protection purposes. Alternatively, the required frost protection could be provided by means of a combination of earth cover and extruded polystyrene insulation. Details on foundation insulation could be provided, if required.

6.5.4 Seismic Site Classification

Site Class D should be used for preliminary design of structures which are founded on the native deposits of silty clay, clayey silt, and glacial till in Areas 2 and 3. Site Class C may also be appropriate, depending on the depth of the bedrock.

In our opinion the soils encountered in the previous boreholes and test pits advanced at the site are not considered to be liquefiable or collapsible under seismic loads.

6.5.5 Foundation Backfill and Drainage

The native soil deposits at this site are highly frost susceptible and should not be used as backfill against foundations, piers, etc. To avoid frost adhesion and possible heaving, the foundations should be backfilled with imported, free-draining, non-frost susceptible granular material meeting OPSS Granular B Type I or II requirements. Where the backfill will ultimately support areas of hard surfacing (pavement, sidewalks or other similar surfaces), the backfill should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment. Where future landscaped areas will exist next to the proposed structure and if some settlement of the backfill is acceptable, the backfill could be compacted to at least 90 percent of the standard Proctor maximum dry density value. In landscaped areas, it may also be possible to backfill the foundation walls with native soils, provided that a bond break is installed on the foundation walls to prevent frost adhesion and heaving.

Where areas of hard surfacing (concrete, sidewalk, pavement, etc.) abut the proposed building, a gradual transition should be provided between those areas of hard surfacing underlain by non-frost susceptible granular wall backfill and those areas underlain by existing frost susceptible native materials to reduce the effects of differential frost heaving. It is suggested that granular frost tapers be constructed from the bottom of the excavation or 1.5 metres below finished grade, whichever is less, to the underside of the granular base/subbase material for the hard surfaced areas. The frost tapers should be sloped at 1 horizontal to 1 vertical, or flatter.

Perimeter foundation drainage is not considered necessary for slab on grade structures at this site provided that the floor slab level is above the finished exterior ground surface level at the building.

6.5.6 Slab-on-Grade Support (Heated Areas Only)

To prevent long term settlement of the floor slabs, all fill material, topsoil, organic, loose, wet or deleterious material should be removed from below the slab on grade.

The grade within the proposed building could be raised, where necessary, with granular material meeting OPSS requirements for Granular B Type I or II. The use of Granular B Type II is preferred under wet conditions. The granular base for the proposed slab on grade should consist of at least 150 millimetres of OPSS Granular A.

OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular A material. Since the source of recycled material cannot be determined, it is suggested that any granular materials used beneath the floor slabs be composed of virgin material (100 percent crushed rock) or native pit run material only for environmental reasons.

All imported granular materials placed below the proposed floor slabs should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density value.

Underfloor drainage is not considered necessary provided that the floor slab level is above the finished exterior ground surface level.

Where any interior areas of the buildings will be unheated, thermal protection for the subgrade will be required where less than 1.5 metres of non-frost susceptible fill cover will exist below the floor slab. Further details on the insulation requirements could be provided, if necessary.

Proper moisture protection with a vapour retarder should be used for any slab on grade where the floor will be covered by moisture sensitive flooring material or where moisture sensitive equipment, products or environments will exist. The “Guide for Concrete Floor and Slab Construction”, ACI 302.1R-04 should be considered for the design and construction of vapour retarders below the floor slab.

6.6 Site Services

6.6.1 Bedding and Cover Materials

The bedding for the sanitary sewers, storm sewers and watermains in overburden should be in accordance with Ontario Provincial Standard Drawing (OPSD) 802.010/802.013 and 802.031/802.033 for flexible and rigid pipes, respectively, for Type 3 soils and bedrock. The pipe bedding should consist of at least 150 millimetres of well graded crushed stone meeting Ontario Provincial Standard Specification (OPSS) for Granular A. OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular A material. Since the source of recycled material cannot be determined, it is suggested that any granular materials used in the service trenches be composed of virgin (i.e., not recycled) material only.

Allowance should be made for subexcavation of any existing fill, organic deposits or disturbed material encountered at subgrade level. In areas where grey silty clay or clayey silt is encountered in the bottom of the excavation, we suggest that the excavation and final trimming to subgrade level be carried out with a shovel equipped with a flat blade bucket.

Allowance should be made to place a subbedding layer composed of 150 to 300 millimetres of OPSS Granular B Type II in areas where wet silt, sandy silt, or silty sand is encountered at the pipe subgrade level to reduce the potential for disturbance.

Cover material, from pipe spring line to at least 300 millimetres above the top of the pipe, should consist of granular material, such as OPSS Granular A.

The use of clear crushed stone should not be permitted on this project, since it could exacerbate groundwater lowering of the overburden materials due to “French Drain” effects.

The subbedding, bedding and cover materials should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density using suitable vibratory compaction equipment.

6.6.2 Trench Backfill

The general backfilling procedures should be carried out in a manner that is compatible with the future use of the area above the service trenches.

In areas where the service trench will be located below or in close proximity to existing or future roadway areas, acceptable native materials should be used as backfill between the roadway subgrade level and the depth of seasonal frost penetration in order to reduce the potential for differential frost heaving between the area over the trench and the adjacent section of roadway. Where native backfill is used, it should match the native materials exposed on the trench walls. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type I. The depth of frost penetration in areas that are kept clear of snow and where the trench backfill consists of broadly graded shattered rock fill or earth fill is expected to be about 1.8 metres. It is our experience, however, that the frost penetration can be as much as 2.4 metres when the trench backfill consists solely of relatively open graded rock fill. Where cover requirements are not practicable, the pipes could be protected from frost using a combination of earth cover and insulation. Further details regarding insulation could be provided, if required.

It is anticipated that most of the inorganic overburden materials encountered during the previous subsurface investigations at the site will be acceptable for reuse as trench backfill. Topsoil or other organic material should be wasted from the trench. If on-site blast rock is used as backfill within the service trench, it should be mostly 300 millimetres, or smaller, in size and should be well graded. The upper surface of the rock fill should be covered with a thin layer of well graded crushed stone to prevent ingress of fine material into voids in the blast rock

To minimize future settlement of the backfill and achieve an acceptable subgrade for the roadways, curbs, driveways, etc., the trench backfill should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. Rock fill should be placed in maximum 500 millimetre thick lifts and compacted with a large drum roller, the haulage and spreading equipment, or a combination of both. The specified density for compaction of the backfill materials may be reduced where the trench backfill is not located below or in close proximity to existing or future areas of hard surfacing and/or structures.

Most of the overburden deposits have water contents that are greater than optimum for compaction. Furthermore, most of the overburden deposits at this site are sensitive to changes in moisture content. Unless these materials are allowed to dry, the specified densities will not likely be possible to achieve and, as a consequence, some settlement of these backfill materials could occur. Consideration could be implementing one or a combination of the following measures to reduce post construction settlement above the trenches, depending on the weather conditions encountered during the construction:

- Allow the overburden materials to dry prior to compaction.
- Reuse any wet materials in the lower part of the trenches and make provision to defer final paving of any roadways for 6 months, or longer, to allow some the trench backfill settlement to occur and thereby improve the final roadway appearance.
- Reuse any wet materials outside hard surfaced areas and where post construction settlement is less of a concern (such as landscaped areas).

6.6.3 Seepage Barriers

The granular bedding in the service trench could act as a “French Drain”, which could promote groundwater lowering. As such, we suggest that seepage barriers be installed along the service trenches at strategic locations at a horizontal spacing of 100 metres, or less. The seepage barriers should begin at subgrade level and extend vertically through the granular pipe bedding and granular surround to within the native backfill materials, and horizontally across the full width of the service trench excavation. The seepage barriers could consist of 1.5 metre wide dykes of compacted weathered silty clay. The weathered silty clay should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. The locations of the seepage barriers could be provided as the design progresses.

6.7 Stormwater Management Pond

6.7.1 Excavation and Temporary Haul Roads

A stormwater management pond is proposed within the northeast part of the site. Based on the results of the previous boreholes and test pits, excavation for the proposed stormwater management pond will likely be required through surficial deposits of topsoil, silty clay, and possibly glacial till.

No unusual constraints are expected in excavating the silty clay overburden material. Groundwater inflow should be expected from the sides and bottom of the excavations, and should be controlled, along with surface water, by pumping from within the excavations or by draining to a sump pit or outlet. As discussed previously, significant groundwater inflow could occur from fissures in the silty clay and existing agricultural tile drains.

The main constraint to excavation will be equipment mobility on the sensitive silty clay deposits. The silty clay soils at this site are sensitive to disturbance and have high water contents. As such, excavation and removal of soil, including trimming to final grade, should be carried out from existing ground surface, if possible. To facilitate excavation and haulage, the excavation could be planned during the winter months on frozen haul roads. It is suggested that temporary haul roadways constructed at or above the existing ground surface on unfrozen overburden material consist of a relatively thick layer of granular material (say 500 millimetres, or more) of Granular B Type II or well shattered and graded blast rock. A woven geotextile separator meeting OPSS 1860 Class II requirement (such as Linq 150EX) is suggested between the native deposits and the granular materials. If conditions warrant, a thicker layer of granular material or blast rock should be used to improve the haul road performance. For haul roads on or within the grey silty clay, an allowance of 600 to 750 millimetres, or more, of OPSS Granular B Type II or well graded blast rock should be made together with a suitable woven geotextile separator.

Topsoil placement on the sides of the cells could be carried out after a period of drying during the summer. Spreading of topsoil will likely require light, tracked equipment (i.e. wide track D3 dozer or smaller).

6.7.2 Long Term Side Slopes

The following site slopes could be used for preliminary design purposes:

Table 6.4 – Summary of Side Slope Inclination

Slope Height	Preliminary Side Slope Inclination
Up to 4 metres	2.5 H to 1 V, or flatter
4 to 8 metres	3.0 H to 1 V, or flatter

6.7.3 Slope Drainage Blanket and Erosion Control

Based on our experience, 2.5 horizontal to 1 vertical, or flatter, excavated side slopes could be protected against erosion using topsoil and seed. If topsoil is to be placed on the bottom and sides of the proposed pond, consideration should be given to carrying out this work using light, track mounted equipment (i.e. wide track D3 dozer or smaller) after a period of prolonged drying over the summer period. To reduce erosion during the development of vegetative cover (say for example along the lower part of the slope above the water level), consideration could be given to temporarily protecting the slopes with a layer of mulch or a photodegradable erosion control blanket, such as those manufactured by North American Green.

6.7.4 Short and Long Term Groundwater Inflow to the Proposed Pond

Excavation for the proposed stormwater management pond will be carried out through deposits of topsoil, silty clay, and possibly glacial till. Groundwater flow should be expected into the pond, both under short and long term conditions.

Under long term conditions, based on our previous experience at other stormwater facilities within silty clay and glacial till deposits in the Ottawa area, the groundwater inflow from the weathered and grey silty clay deposits is expected to be relatively small.

Deep boreholes should be advanced in the area of the proposed pond to assess the potential for basal uplift and groundwater inflow from the bedrock.

6.7.5 Effects of the Proposed Pond on Nearby Structures and Services

As indicated in Section 6.5.4, groundwater inflow into the proposed stormwater management pond should be expected in the short and long term. Some minor and localized groundwater level lowering could occur in close proximity to the proposed pond as a result of normal gravity flow of groundwater toward the cells. The zone of influence of the groundwater lowering should be relatively small (less than about 20 metres) from the edge of the proposed pond constructed in silty clay. The zone of influence and the amount of groundwater inflow could be significant if the soil deposits are underlain at shallow depth by bedrock.

6.7.6 Permanent Access Roadways

In preparation for the construction of the permanent, granular access roadway at the proposed stormwater management pond, all topsoil, organic material, fill material, and loose and disturbed soil should be removed from the subgrade surface. The subgrade should then be proof rolled with a large steel drum roller. Any soft areas evident from the proof rolling should be subexcavated and replaced with compacted earth borrow material.

The following minimum granular thicknesses should be used for preliminary design in areas where the roadway construction will be carried out within the table land area at or above existing ground surface,

- 150 millimetres of OPSS Granular A, over
- 450 millimetres of OPSS Granular B Type II (100 millimetre minus crushed stone), over
- Woven geotextile separator meeting OPSS 1860 Class II
- Native, undisturbed weathered silty clay or glacial till

Where the access roadways are in cut below existing ground surface, the thickness of the Granular B Type II should be increased to between 600 and 750 millimetres, depending on the subgrade conditions.

The above granular thicknesses assume that the subgrade surface is not disturbed. If the roadway subgrade surface is disturbed or wetted due to construction operations or precipitation, the granular thicknesses given above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase. The adequacy of the design pavement thickness should be assessed by geotechnical personnel at the time of construction.

The granular materials should be placed and compacted in maximum 300 millimetre thick lifts to at least 98 percent of the standard Proctor dry density value.

In areas where heavy maintenance vehicles will be required in the bottom of the pond, allowance should be made for the following minimum granular thickness:

- 600 to 750 millimetres of OPSS Granular B Type II, over
- Woven geotextile separator meeting OPSS 1860 Class II
- Native, undisturbed silty clay

The above granular thickness assumes that the subgrade surface is not disturbed due to excavation and water.

The granular material should be placed under dry conditions and should be compacted in one lift to at least 95 percent of the standard Proctor dry density value.

Some distortion of the access roadway could occur due to freezing and thawing of the subgrade materials during the winter period. In addition, some unavoidable erosion of fines from the gravel should be expected. As such, allowance should be made for relevelling the gravel access roadway as the need arises.

6.7.7 Asphaltic Concrete Surfaced Access Pathways

In preparation for the construction of the permanent access pathways, all topsoil, organic material, fill material, and loose and disturbed soil should be removed from the subgrade surface. The subgrade should then be proof rolled with a large steel drum roller. Any soft areas evident from the proof rolling should be subexcavated and replaced with compacted earth borrow material.

Permanent, asphaltic concrete surfaced pathways located within the project area should be constructed as per the City of Ottawa standard detail drawing SC20. The following minimum asphaltic concrete and granular thicknesses should be used for pedestrian pathways:

- 50 millimetres of OPSS SP12.5 asphaltic concrete, over
- 150 millimetres of OPSS Granular A, over
- 200 millimetres of OPSS Granular B Type II
- Non-woven geotextile separator meeting OPSS 1860 Class I (when warranted)
- Native, undisturbed weathered silty clay

Permanent, asphaltic concrete surfaced pathways that are to be used by pedestrian and light service vehicles could be constructed using the following minimum granular thicknesses:

- 50 millimetres of OPSS SP12.5 asphaltic concrete, over
- 150 millimetres of OPSS Granular A, over
- 300 millimetres of OPSS Granular B Type II (100 millimetre minus crushed stone)
- Woven geotextile separator meeting OPSS 1860 Class I (when warranted)
- Native, undisturbed weathered silty clay

The above granular thicknesses assume that the subgrade surface is relatively undisturbed. If the subgrade surface is disturbed or wetted due to construction operations or precipitation, the granular thicknesses given above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase and/or to incorporate a geotextile separator between the subgrade surface and the granular subbase material. The adequacy of the design pavement thickness should be assessed by geotechnical personnel at the time of construction.

The granular materials should be placed and compacted in maximum 300 millimetre thick lifts to at least 98 percent of the standard Proctor dry density value.

6.8 Roadways

6.8.1 Subgrade Preparation

In preparation for roadway construction at this site, all surficial topsoil and any soft, wet or deleterious materials should be removed from the proposed roadway areas. This would include the removal of any organic material and/or disturbed soil along the existing agricultural drains.

Prior to placing granular material for the roadway, the exposed subgrade should be heavily proof rolled with a large (10 tonne) vibratory steel drum roller under dry conditions and inspected and approved by geotechnical personnel. Any soft areas evident from the proof rolling should be subexcavated and replaced with suitable (dry) earth borrow or well shattered and graded rock fill material that is frost compatible with the materials exposed on the sides of the area of subexcavation.

Similarly, should it be necessary to raise the roadway grades at this site, material which meets OPSS specifications for Select Subgrade Material, earth borrow or well shattered and graded rock fill material may be used. In low, wet areas, well shattered and graded rock fill material is preferred.

The select subgrade material or earth borrow should be placed in maximum 300 millimetre thick lifts and compacted to at least 95 percent of the standard Proctor maximum dry density value using vibratory compaction equipment. Rock fill should also be placed in thin lifts and suitably compacted either with a large drum roller, the haulage and spreading equipment, or a combination of both.

It is our experience that the upper part of the weathered silty clay (i.e., within 0.3 to 0.5 metres from original ground surface) may be impacted by past frost action. During removal of the topsoil and fill material, the upper part of the silty clay could unavoidably peel upwards and become disturbed. Where this occurs in the access roadway areas, the upper part of the silty clay should be re-compacted in place using suitable compaction equipment.

Truck traffic should be avoided on the native soil subgrade, especially under wet conditions.

6.8.2 Pavement Structure for Roadways

The following preliminary pavement structures could be used planning purposes:

Internal Local Road:

> 100 millimetres of hot mix asphaltic concrete (40 millimetres of Superpave 12.5 (Traffic Level A or B) over 60 millimetres of Superpave 19.0 (Traffic Level A or B)), over
> 150 millimetres of OPSS Granular A base, over
> 375 millimetres of OPSS Granular B, Type II subbase

Minor Collector Roads:

100 millimetres of hot mix asphaltic concrete (40 millimetres of Superpave 12.5 (Traffic Level A or B) over 60 millimetres of Superpave 19.0 (Traffic Level A or B)), over
150 millimetres of OPSS Granular A base, over
450 to 500 millimetres of OPSS Granular B, Type II subbase

North South Aligned Arterial:

130 millimetres of hot mix asphaltic concrete (50 millimetres of Superpave FC1 or FC2 (Traffic Level D over 80 millimetres of Superpave19.0 (Traffic Level D)), over
150 millimetres of OPSS Granular A base, over
600 to 700 millimetres of OPSS Granular B, Type II subbase

6.8.3 Pedestrian and Light Service Vehicle Pathways

A pedestrian pathway may be constructed along the proposed arterial roadway. It is expected that occasional light trucks will use the pathway for service purposes.

The pathway should be constructed with the following minimum pavement structure:

50 millimetres of hot mix asphaltic concrete (Superpave 12.5 (Traffic Level A or B), over
150 millimetres of OPSS Granular A base, over
300 millimetres of OPSS Granular B Type II subbase

The above pavement structures are preliminary and should be verified by the geotechnical engineer once projected Annual Average Daily Traffic (AADT) data become available.

The OPSS Granular B Type II subbase material could be reduced in areas where bedrock is encountered, and where sand and gravel or well graded and compacted rock fill is used to raise the grade below the roadway.

In areas where bedrock or well shattered and graded rock fill is encountered at the pavement subgrade level, the thickness of the OPSS Granular B Type II subbase could be reduced to 150 millimetres.

6.8.4 Effects of Soil Disturbance

The above pavement structures assume that any trench backfill is adequately compacted and that the roadway subgrade surface is prepared as described in this report. If the roadway subgrade surface is disturbed or wetted due to construction operations or precipitation, the granular thickness given above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase and/or to incorporate a woven geotextile separator between the roadway subgrade surface and the granular subbase material. The adequacy of the design pavement thickness should be assessed by geotechnical personnel at the time of construction. In our experience, a woven geotextile and additional subbedding material (for a total of about 600 millimetres of OPSS Granular B Type II) will likely be required in most cases where the subgrade consists of overburden, if the roadway construction is planned during the wet period of the year (such as the early spring or fall).

Similarly, if the granular pavement materials are to be used by construction traffic, it may be necessary to increase the thickness of the Granular B Type II, install a woven geotextile separator between the roadway subgrade surface and the granular subbase material, or a combination of both, to prevent pumping and disturbance to the subbase material. The contractor should be made responsible for their construction access.

6.8.5 Granular Material Compaction

The pavement granular materials should be compacted in maximum 300 millimetre thick lifts to at least 98 percent of standard Proctor maximum dry density using suitable vibratory compaction equipment.

6.8.6 Asphaltic Concrete Types

The asphaltic concrete should consist of 40 millimetres of Superpave 12.5 over one 60 to 80 millimetre lift of Superpave 19.0. Performance grade PG 58-34 asphaltic cement should be specified.

For the arterial roadway, the asphaltic concrete should consist of a 50 millimetre surface layer of Superpave FC1 or FC2 (Traffic Level D) over one 80 millimetre thick layers of Superpave 19.0 (Traffic Level D). Performance grade PG 64-34 asphalt should be considered.

6.8.7 Transition Treatments and Frost Tapers

Where the new pavement structures will abut the existing pavements, the depths of the granular materials should taper up or down at 5 horizontal to 1 vertical, or flatter, to match the depths of the granular material(s) exposed in the existing pavement.

Granular frost tapers should be installed in accordance with OPSD 205.030 in areas where there is an abrupt transition from bedrock to overburden.

6.8.8 Pavement Drainage

The subgrade surface should be shaped and crowned to promote drainage of the roadway granular materials.

Adequate drainage if the pavement granular materials and subgrade is important for the long term performance of the pavement at this site. As such it is recommended that catch basins be provided with perforated stub drains extending about 3 metres out from the catch basins in two directions parallel to the roadway. These drains should be installed at the bottom of the subbase layer.

6.9 Other Considerations

6.9.1 Construction Induced Vibration

Some of the construction operations (such as bedrock removal by blasting or hoe ramming, granular material compaction, excavation, foundation construction etc.) will cause ground vibration on and off of the site. The vibrations will attenuate with distance from the source, but may be felt at nearby structures. Provided that good construction practices are used, the magnitude of the vibrations will be much less than that required to cause damage to the nearby structures or services, but may be felt at the nearby structures. We recommend that preconstruction surveys be carried out on the adjacent structures and that vibration monitoring be carried out during the construction.

6.9.2 Winter Construction

In the event that construction is required during freezing temperatures, the soil below the footings should be protected immediately from freezing using straw, propane heaters and insulated tarpaulins, or other suitable means.

Any service trenches should be opened for as short a time as practicable and the excavations should be carried out only in lengths which allow all of the construction operations, including backfilling, to be fully completed in one working day. The materials on the sides of the trenches should not be allowed to freeze. In addition, the backfill should be excavated, stored and replaced without being disturbed by frost or contaminated by snow or ice.

6.9.3 Excess Soil Management Plan

This report does not constitute an excess soil management plan. The disposal requirements for excess soil from the site have not been assessed.

6.9.4 Landscape Design

Based on the results of the geotechnical investigation, portions of the site are underlain by deposits of silty clay, a material which is known to be susceptible to shrinkage with a change/reduction in moisture content. Research by the Institute for Research in Construction (formerly the Division of Building Research) of the National Research Council of Canada has shown that trees can cause a reduction of moisture content in the silty clay soils in the Ottawa area, which can result in significant settlement/damage to nearby buildings supported on shallow foundations bearing on or within these deposits. To minimize the potential for this, the separation distance between deciduous trees and houses, which are founded on or within silty clay deposits, should be greater than the ultimate height of the tree. For multiple trees, the separation distance should be increased to 1.5 times the trees' ultimate height.

The landscape design plan should take into account the effects of all existing and future trees on structures, roads/hard surfaced areas, and services.

6.9.5 Stability of Slopes

Based on a site reconnaissance, there are no slopes at the site that would be of concern from a slope stability point of view.

7.0 ADDITIONAL INVESTIGATION

This preliminary geotechnical investigation is based on widely spaced test pits and boreholes together with available subsurface information and is intended for planning purposes only. It is recommended that detailed geotechnical investigations be conducted as the design progresses.

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report, please do not hesitate to contact our office.

Andrew Chevrier, M.Eng.,P.Eng.
Senior Geotechnical Engineer

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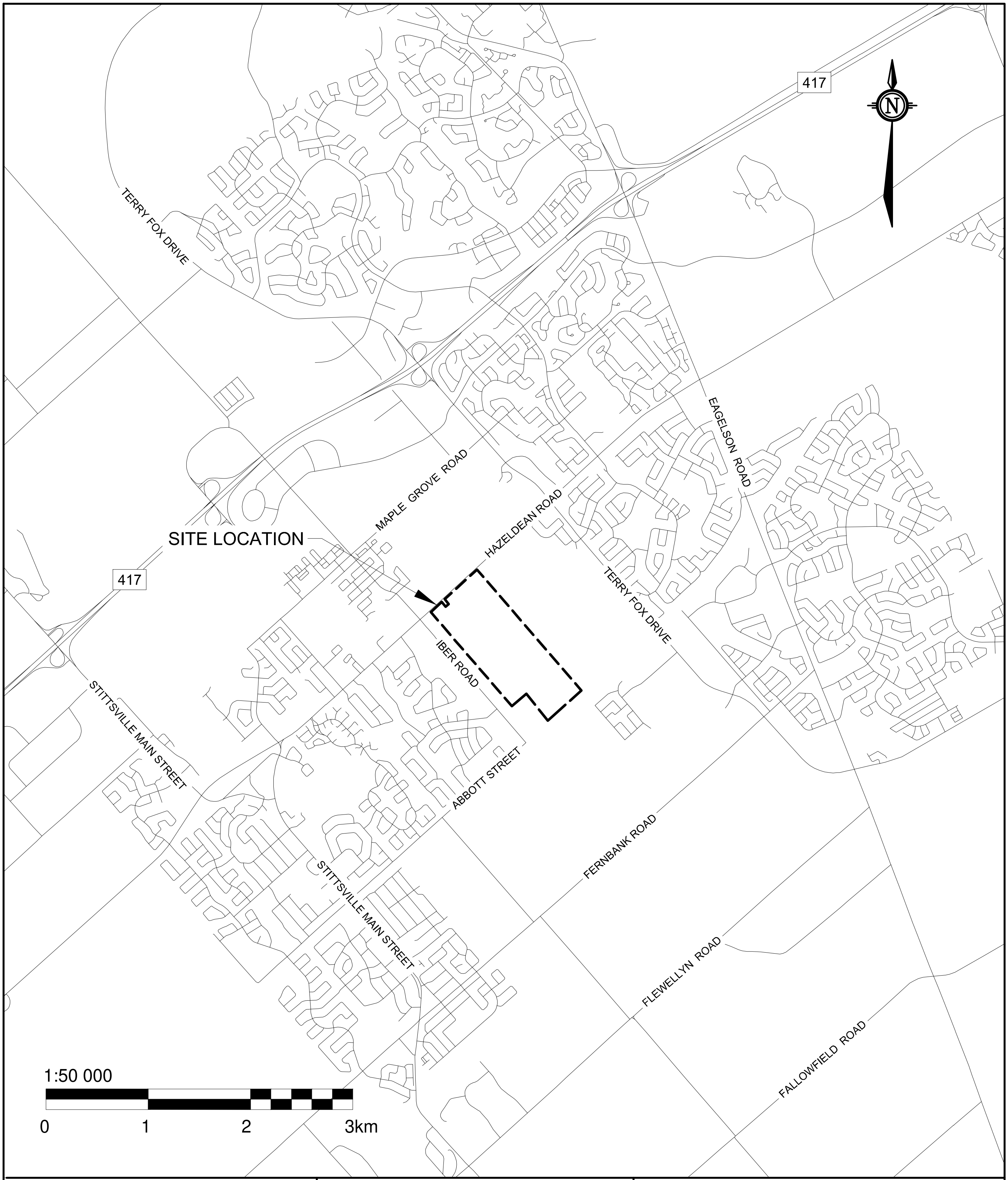
25 Aug 2016
PROVINCE OF ONTARIO



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Project

GEOTECHNICAL OVERVIEW
KIZELL LANDS
OTTAWA, ONTARIO

Drawing

KEY PLAN

Drwn By	Chkd By	Date	Project No.	Revision No.	
P.C.	A.F.C.	DECEMBER 2015	64153.50	0	**FIGURE 1**

LEGEND

TP #
APPROXIMATE TEST PIT LOCATION IN PLAN
(previous investigation by Houle Chevrier Engineering Ltd., Report # 06-384, May 2007.
- See Appendix B)

BH #
APPROXIMATE BOREHOLE LOCATION IN PLAN
(previous investigation by Houle Chevrier Engineering Ltd. Report # 06-384, May 2007.
- See Appendix B)

BH #
APPROXIMATE BOREHOLE LOCATION IN PLAN
(previous investigation by Houle Chevrier Engineering Ltd. Report # 12-247, September 4, 2013.
- See Appendix C)

BH #
APPROXIMATE BOREHOLE LOCATION IN PLAN
(previous investigation by Golder Associates Ltd. Report # 001-2225. - See Appendix D)

BH #
APPROXIMATE BOREHOLE LOCATION IN PLAN
(previous investigation by Jaques Whitford Ltd. Report # ONO11725. - See Appendix E)

AREA # REFER TO ACCOMPANYING REPORT FOR AREA DESCRIPTIONS. BOUNDARIES BETWEEN AREAS ARE BASED ON WIDELY SPACED TEST HOLES AND SHOULD BE CONSIDERED INFERRED AND APPROXIMATE.

NOTE: INFORMATION ON THIS PLAN SHOULD NOT BE USED FOR DESIGN OR CONSTRUCTION AT ANY SPECIFIC LOCATIONS, NOR SHOULD IT BE USED AS A REPLACEMENT FOR SITE-SPECIFIC GEOTECHNICAL INVESTIGATIONS.

Scale 1:7500

0 150 300 450m

Houle Chevrier Engineering

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Drawing
SITE PLAN

Client
NOVATECH

Project
64153.50

GEOTECHNICAL OVERVIEW
KIZELL LANDS
OTTAWA, ONTARIO

Drawn by
P.C.

Checkd by
A.F.C.

Date
DECEMBER 2015

Rev.
0

FIGURE 2

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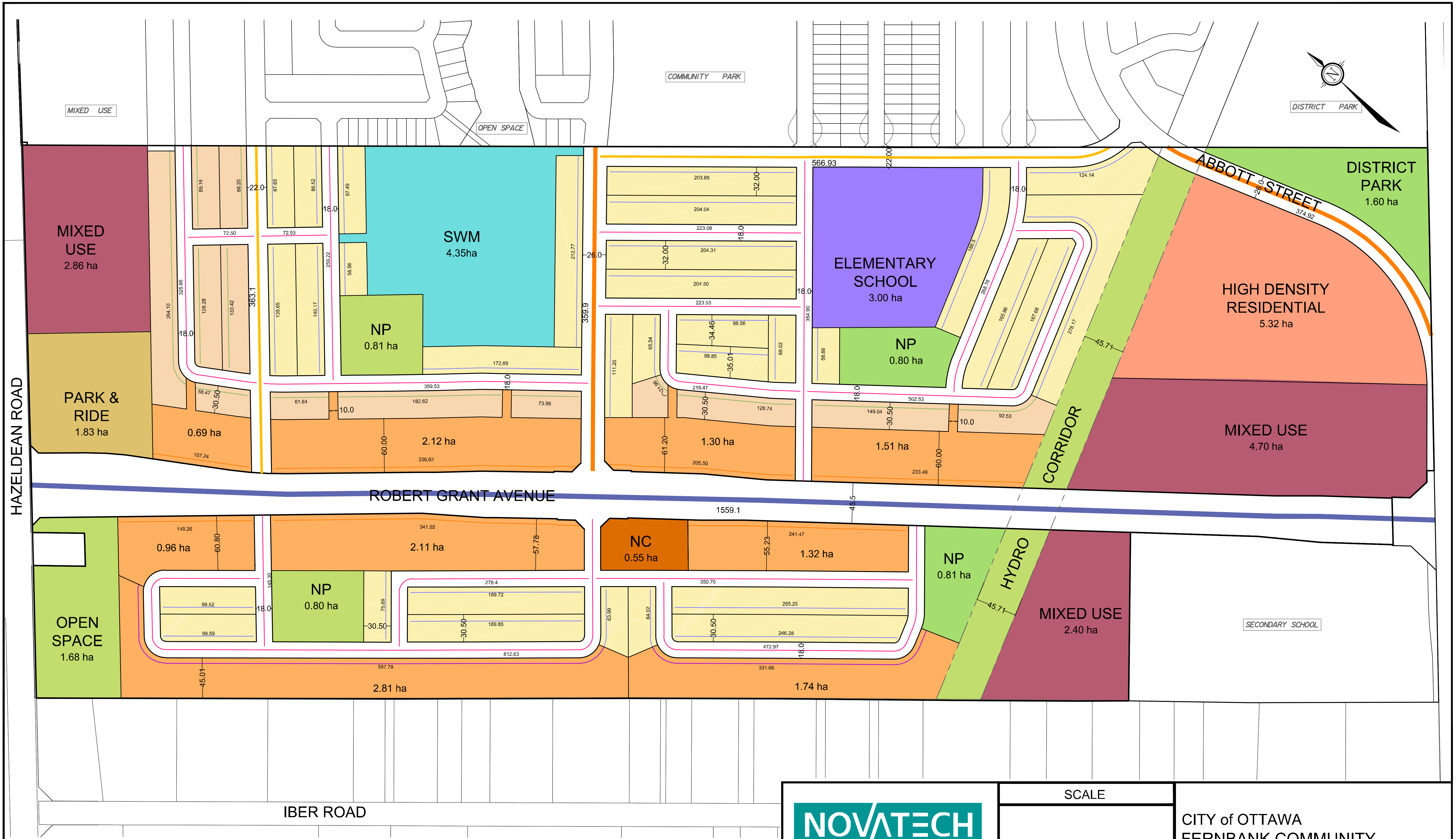
APPENDIX A

Concept Plan for Kizell Property



ROAD LENGTH BREAKDOWN	m	ft
🟦 Arterial	1,559.1	5,115.1
🟧 Major Collector	734.8	2,410.8
🟨 Minor Collector	930.0	3,051.2
— 18.0m R.O.W	4,941.7	16,212.8
Total Road Lengths	8,165.6	26,789.9

SALEABLE FRONTAGE	m	ft	ha	Acres
— Singles	4,519.4	14,827.3	14.87	36.74
— Townhomes	1,472.1	4,829.8	4.85	11.98
— Stacked Townhomes	929.5	3,049.4	4.55	11.24
— Multi-Family Residential (Fronting on Arterial)	1,614.1	5,295.6	10.02	24.76
Total Residential	8,535.1	28,002.1	34.29	84.73

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SCALE

1:4,000

1:4,000
0 40 80 120 160

CITY of OTTAWA
FERNBANK COMMUNITY

KIZELL LANDS
CONCEPT PLAN 18

JULY 21, 2016 108195-CP18

M:\2008\108195\Subdivision\CAD\Planning\Concept Plans\108195-CP18.dwg, CP-11x17, Jul 21, 2016 - 11:00am, wsloss

SHT11X17.DWG - 279mmX432mm

APPENDIX B

Record of Test Pits and Boreholes
Previous Investigation by Houle Chevrier Engineering Ltd.
(Report Number 06-384)

RECORD OF TEST PIT 1

PROJECT: 06-384

LOCATION: See Site Plan, Figure 5

DATE OF EXCAVATION: November 15, 2006

SHEET 1 OF 1

DATUM: Not Applicable

TYPE OF EXCAVATOR: Hydraulic Excavator

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ 20 40 60 80	WATER CONTENT (PERCENT) Wp ⊢—o W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface							
	TOPSOIL							∇
1	Very stiff to stiff grey brown SILTY CLAY (weathered crust)	▨	0.70					
2								
3	End of test pit Notes: 1. Substantial ground water inflow observed. Test pit filled within a few minutes after excavation.		2.90					Groundwater inflow observed at 0.50 metres below ground surface on November 15, 2006
4								
5								
6								
7								
8								
9								
10								

DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: EG

CHECKED:

PROJECT: 06-384

RECORD OF TEST PIT 2

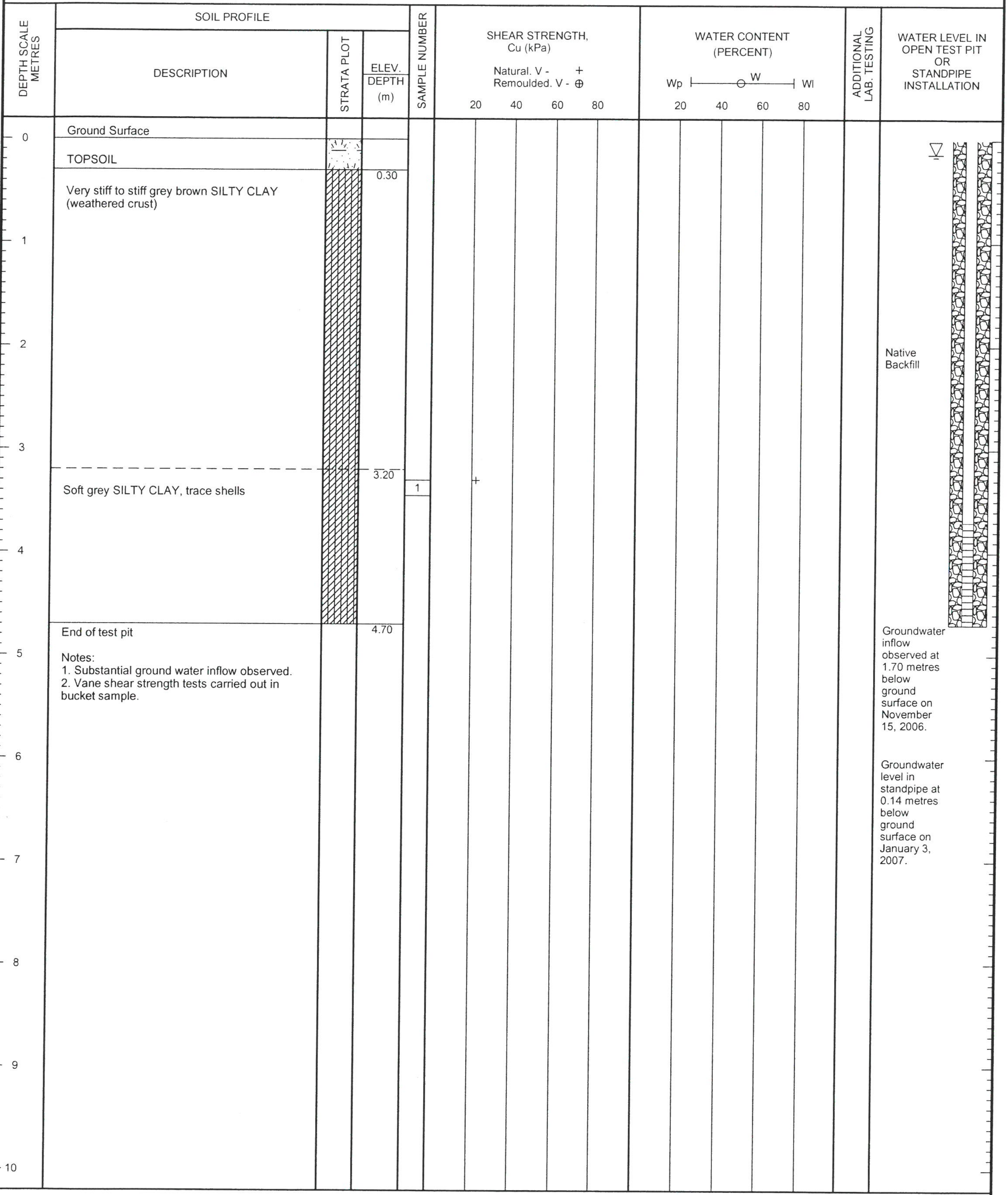
SHEET 1 OF 1

LOCATION: See Site Plan, Figure 5

DATUM: Not Applicable

DATE OF EXCAVATION: November 15, 2006

TYPE OF EXCAVATOR: Hydraulic Excavator



DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa)	WATER CONTENT (PERCENT)	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface							∇
	TOPSOIL							
	Very stiff to stiff grey brown SILTY CLAY (weathered crust)		0.30					Native Backfill
	Soft grey SILTY CLAY, trace shells		3.20	1	+			
	End of test pit Notes: 1. Substantial ground water inflow observed. 2. Vane shear strength tests carried out in bucket sample.		4.70					Groundwater inflow observed at 1.70 metres below ground surface on November 15, 2006. Groundwater level in standpipe at 0.14 metres below ground surface on January 3, 2007.

DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: EG

CHECKED: AC

PROJECT: 06-384

LOCATION: See Site Plan, Figure 5

DATE OF EXCAVATION: November 15, 2006

RECORD OF TEST PIT 3

SHEET 1 OF 1

DATUM: Not Applicable

TYPE OF EXCAVATOR: Hydraulic Excavator

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ 20 40 60 80	WATER CONTENT (PERCENT) Wp ├──○ W ──┤ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface		99.06					
	TOPSOIL	▒	98.66 0.40					
1 2 3	Very stiff to stiff grey brown SILTY CLAY (weathered crust)	▨	95.56 3.50					∇
4 5 6 7 8 9 10	End of test pit Notes: 1. Tile drain broken 0.8m below ground surface causing substantial water inflow. Test pit filled within a few minutes after excavation. 2. Sides of test pit collapsed during excavating.							Groundwater inflow observed at 0.80 metres below ground surface on November 15, 2006

DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: EG

CHECKED: AC

PROJECT: 06-384

LOCATION: See Site Plan, Figure 5

DATE OF EXCAVATION: November 15, 2006

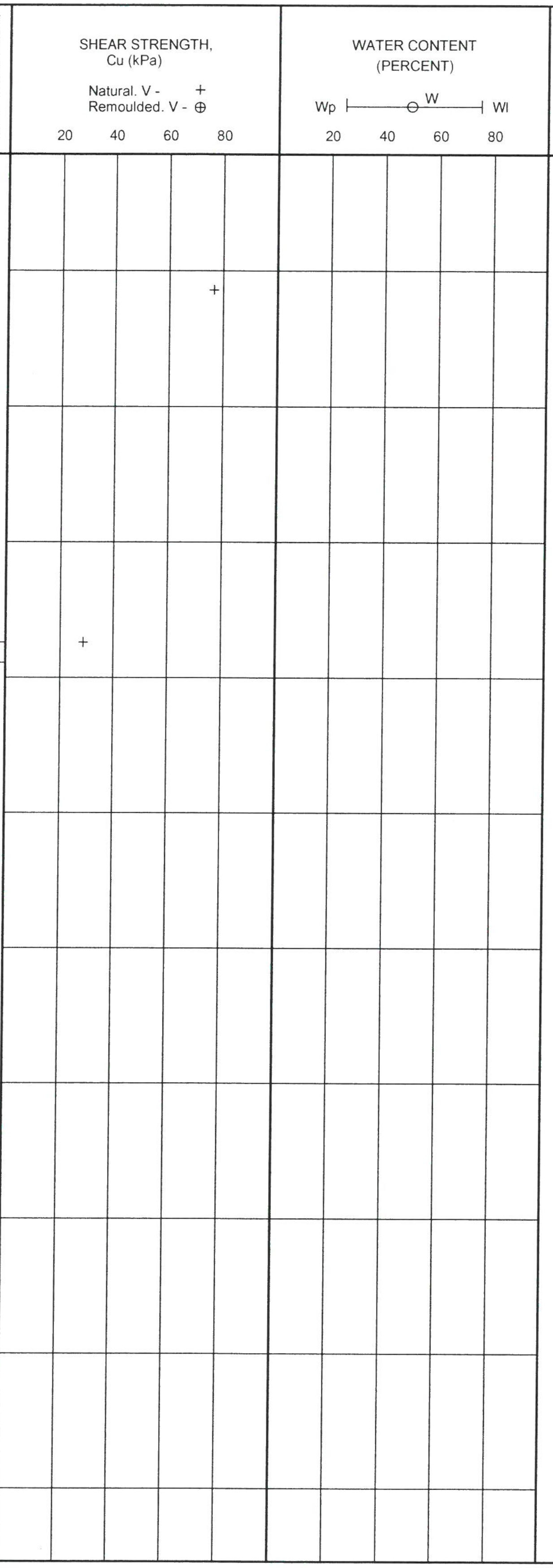
RECORD OF TEST PIT 5

SHEET 1 OF 1

DATUM: Not Applicable

TYPE OF EXCAVATOR: Hydraulic Excavator

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface		101.66			
	TOPSOIL		101.36 0.30			
1	Very stiff to stiff grey brown SILTY CLAY (weathered crust)					
2						∇
3			98.36 3.30			
	Firm grey SILTY CLAY			1		
4	End of test pit		97.66 4.00			Groundwater inflow observed at 2.00 metres below ground surface on November 15, 2006
	Note: Vane shear strength tests carried out in bucket sample					
5						
6						
7						
8						
9						
10						



DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: EG

CHECKED: AC

PROJECT: 06-384

LOCATION: See Site Plan, Figure 5

DATE OF EXCAVATION: November 15, 2006

RECORD OF TEST PIT 6

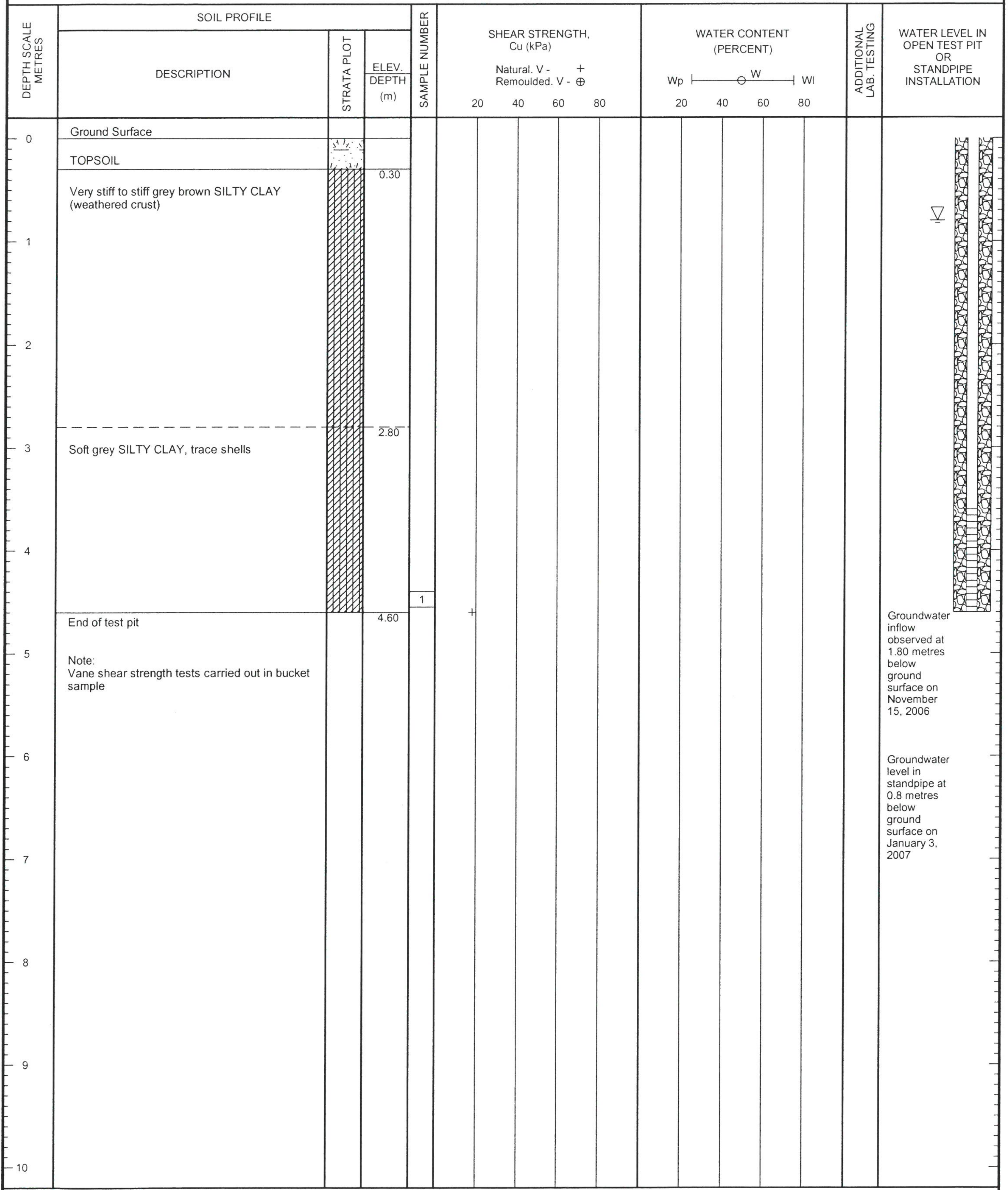
SHEET 1 OF 1

DATUM: Not Applicable

TYPE OF EXCAVATOR: Hydraulic Excavator

SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER
Ground Surface		
TOPSOIL	0.30	
Very stiff to stiff grey brown SILTY CLAY (weathered crust)	2.80	
Soft grey SILTY CLAY, trace shells	4.60	1
End of test pit		

Note:
Vane shear strength tests carried out in bucket sample



Groundwater inflow observed at 1.80 metres below ground surface on November 15, 2006

Groundwater level in standpipe at 0.8 metres below ground surface on January 3, 2007

DEPTH SCALE
1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: EG

CHECKED: AC

PROJECT: 06-384

RECORD OF TEST PIT 9

SHEET 1 OF 1

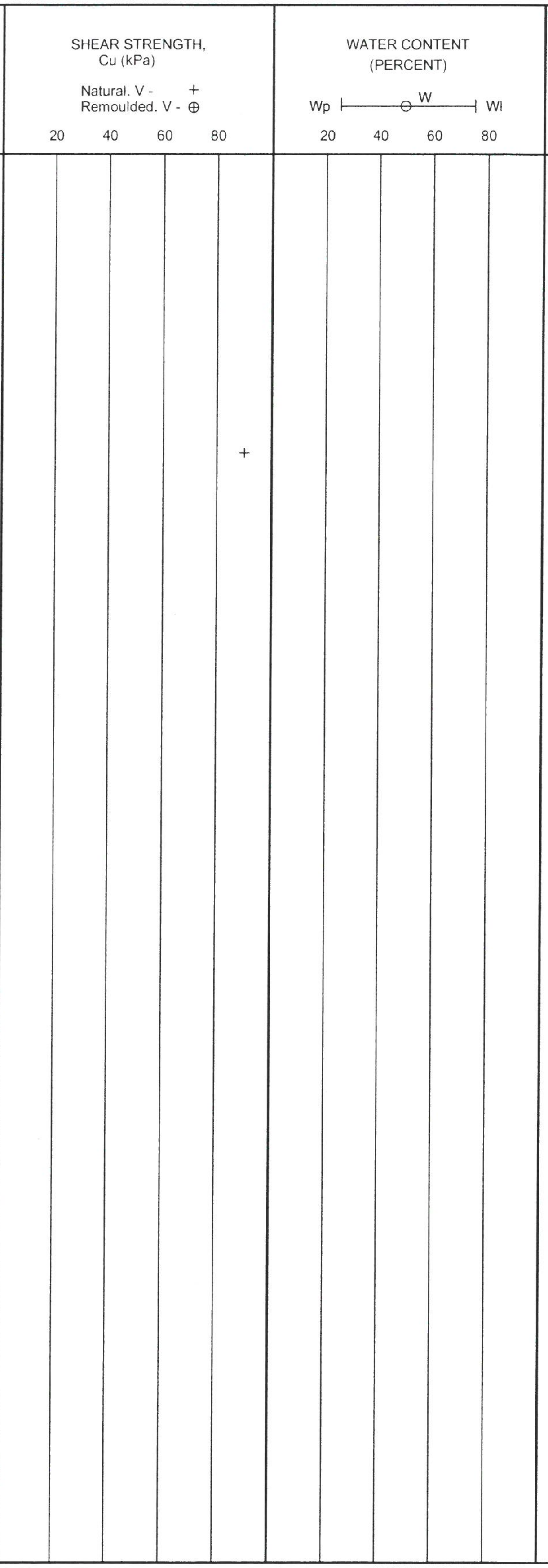
LOCATION: See Site Plan, Figure 5

DATUM: Not Applicable

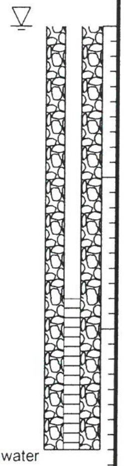
DATE OF EXCAVATION: December 5, 2006

TYPE OF EXCAVATOR: Hydraulic Excavator

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	ADDITIONAL LAB. TESTING
0	Ground Surface				
	TOPSOIL				
	Stiff grey brown SILTY CLAY, trace cobbles (weathered crust)	▨	0.20		
1	Stiff grey SILTY CLAY, some shells	▨	0.90		
2					
3	End of test pit Notes: 1. Substantial ground water inflow observed at ground level. 2. Sides of test pit collapsed during excavating.		2.80		
4					
5					
6					
7					
8					
9					
10					



WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION



Native Backfill

Groundwater inflow observed at ground level on December 5, 2006

Groundwater level in standpipe at ground surface on January 3, 2007.

PROJECT: 06-384

RECORD OF TEST PIT 10

SHEET 1 OF 1

LOCATION: See Site Plan, Figure 5

DATUM: Not Applicable

DATE OF EXCAVATION: November 15, 2006

TYPE OF EXCAVATOR: Hydraulic Excavator

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ 20 40 60 80	WATER CONTENT (PERCENT) Wp ⊢—○ W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface							
	TOPSOIL							
	Very stiff to stiff grey brown SILTY CLAY (weathered crust)		0.25					
1	Grey brown SILT, some fine sand, trace clay		0.88	1				▽
2	Grey brown to brown sandy silt, some gravel, cobbles and boulders (GLACIAL TILL)		1.60					
3								
	End of test pit		3.70					Groundwater inflow observed at 1.50 metres below ground surface on November 15, 2006
4								
5								
6								
7								
8								
9								
10								

TESTPIT_RECORD 06-384 TESTPIT LOGS.GPJ MHECL.GDT 5/28/07

DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: EG

CHECKED: AC

PROJECT: 06-384

RECORD OF TEST PIT 11

SHEET 1 OF 1

LOCATION: See Site Plan, Figure 5

DATUM: Not Applicable

DATE OF EXCAVATION: November 15, 2006

TYPE OF EXCAVATOR: Hydraulic Excavator

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ 20 40 60 80	WATER CONTENT (PERCENT) Wp ⊢—⊖ W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface		100.84					
	TOPSOIL		100.54 0.30					
1 2 3	Very stiff to stiff grey brown SILTY CLAY (weathered crust)		97.04 3.80					∇
4	Grey brown silty sand, some clay, gravel, cobbles and boulders (GLACIAL TILL)		96.34 4.50					
5 6 7 8 9 10	End of test pit							Groundwater inflow observed at 2.50 metres below ground surface on December 5, 2006

TESTPIT_RECORD 06-384 TESTPIT LOGS.GPJ MHECL.GDT 5/28/07

DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: EG

CHECKED: AC

PROJECT: 06-384

RECORD OF TEST PIT 18

SHEET 1 OF 1

LOCATION: See Site Plan, Figure 5

DATUM: Not Applicable

DATE OF EXCAVATION: November 15, 2006

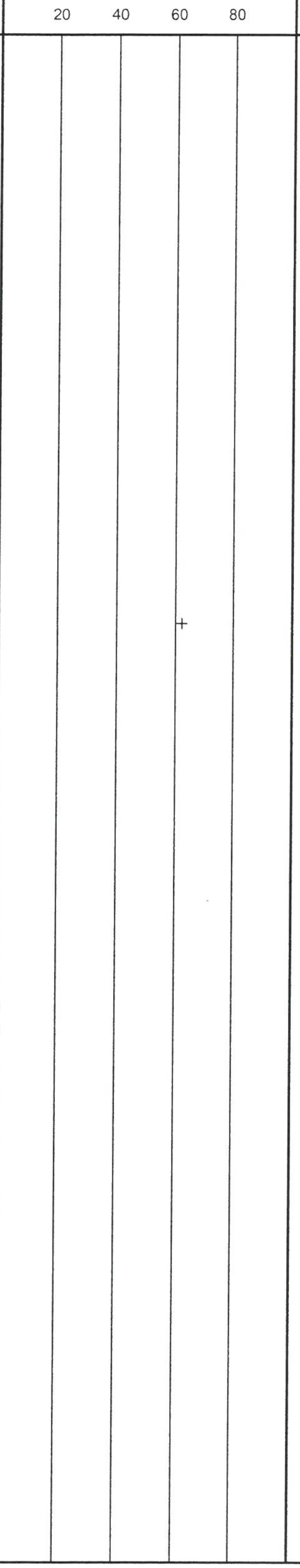
TYPE OF EXCAVATOR: Hydraulic Excavator

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface				
	Dark brown clayey silt (TOPSOIL)	0.25			
	Very stiff to stiff brown SILTY CLAY, trace cobbles (weathered crust)	1.40			
	Stiff grey SILTY CLAY	4.30			Native Backfill
	End of test pit				Groundwater inflow observed at 1.90 metres below ground surface on November 15, 2006 Groundwater level in standpipe at1.60 metres below ground surface on January 3, 2007.

SHEAR STRENGTH, Cu (kPa)

Natural. V - +

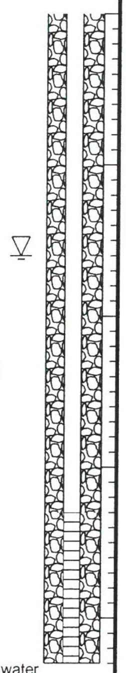
Remoulded. V - ⊕



WATER CONTENT (PERCENT)

Wp ⊢—○ W —⊣ Wl

20 40 60 80



DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: EG

CHECKED: AC

PROJECT: 06-384

RECORD OF TEST PIT 19

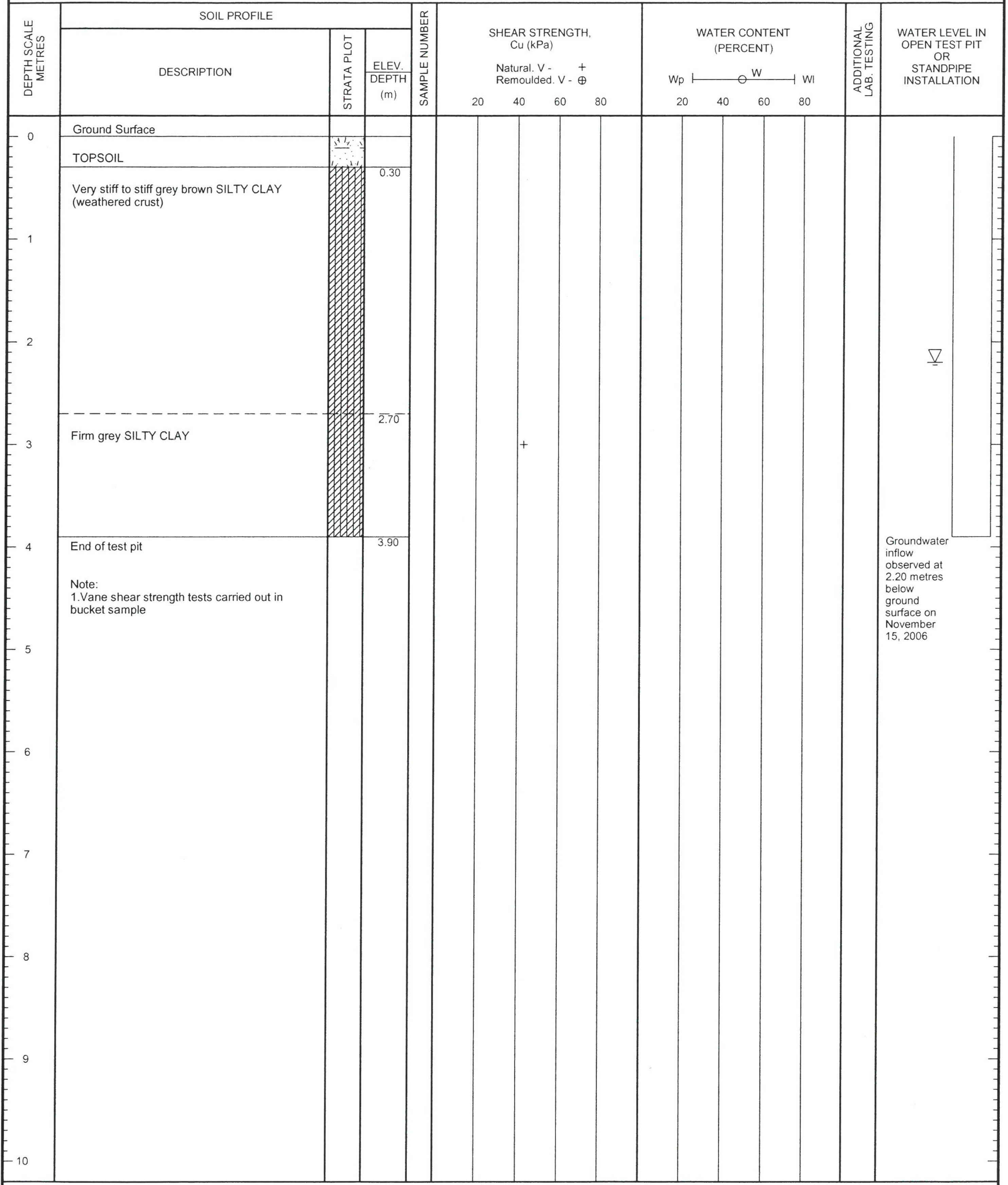
SHEET 1 OF 1

LOCATION: See Site Plan, Figure 5

DATUM: Not Applicable

DATE OF EXCAVATION: November 15, 2006

TYPE OF EXCAVATOR: Hydraulic Excavator



DESCRIPTION	ELEV. DEPTH (m)
Ground Surface	
TOPSOIL	0.30
Very stiff to stiff grey brown SILTY CLAY (weathered crust)	2.70
Firm grey SILTY CLAY	3.90
End of test pit	

Note:
1.Vane shear strength tests carried out in bucket sample

Groundwater inflow observed at 2.20 metres below ground surface on November 15, 2006

DEPTH SCALE
1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: EG

CHECKED: AC

PROJECT: 06-384

RECORD OF BOREHOLE 1

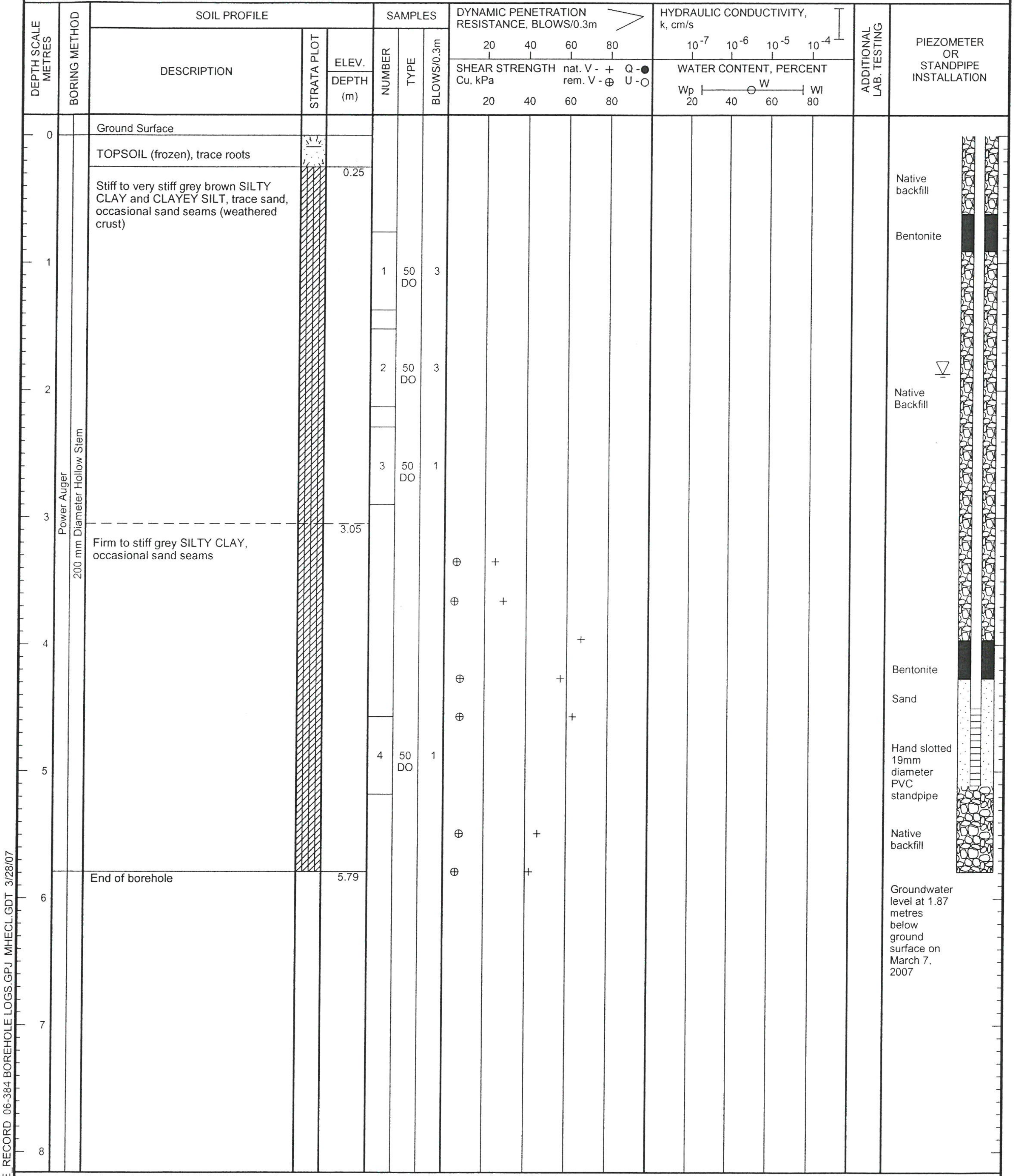
SHEET 1 OF 1

LOCATION: Refer to Site Plan, Figure 1

DATUM:

BORING DATE: February 19, 2007

SPT HAMMER: 63.6 kg; drop 0.76 m



DESCRIPTION	ELEV. DEPTH (m)
Ground Surface	
TOPSOIL (frozen), trace roots	0.25
Stiff to very stiff grey brown SILTY CLAY and CLAYEY SILT, trace sand, occasional sand seams (weathered crust)	3.05
Firm to stiff grey SILTY CLAY, occasional sand seams	5.79
End of borehole	

NUMBER	TYPE	BLOWS/0.3m
1	50 DO	3
2	50 DO	3
3	50 DO	1
4	50 DO	1

Groundwater level at 1.87 metres below ground surface on March 7, 2007

DEPTH SCALE

1 to 40

Houle Chevrier Engineering Ltd.

LOGGED: PA

CHECKED: AC

BOREHOLE RECORD 06-384 BOREHOLE LOGS.GPJ MHECL.GDT 3/28/07

PROJECT: 06-384

RECORD OF BOREHOLE 2

SHEET 1 OF 1

LOCATION: Refer to Site Plan, Figure 1

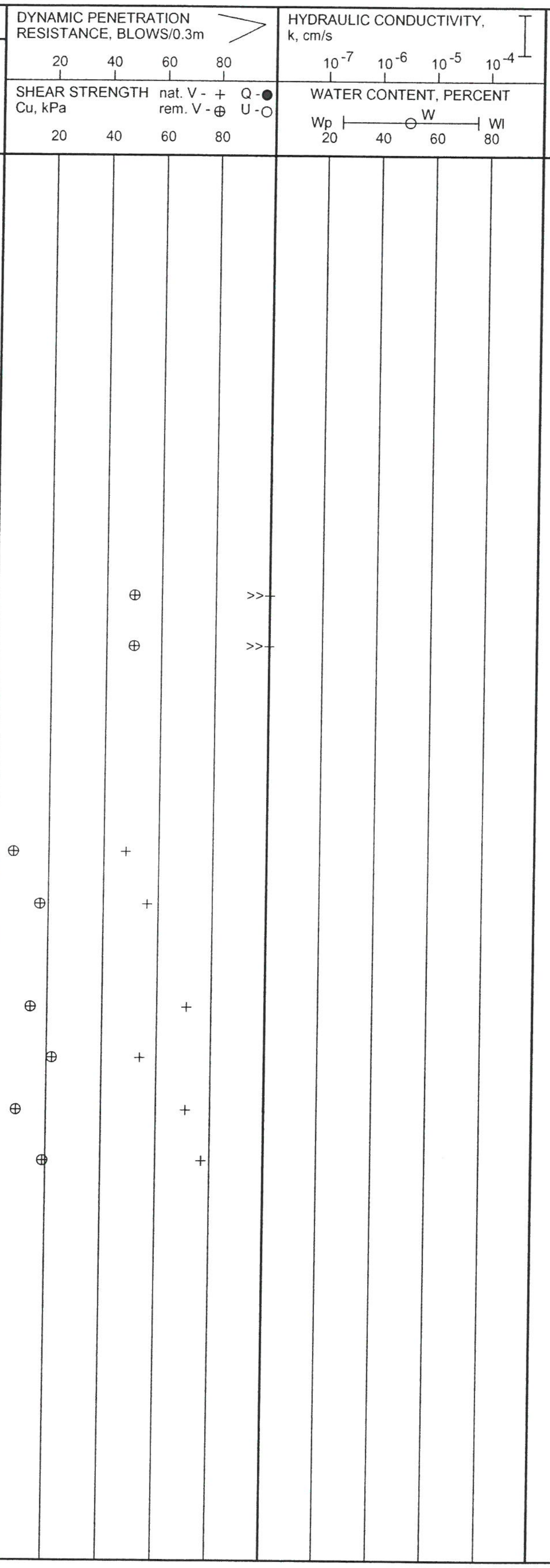
DATUM:

BORING DATE: February 19, 2007

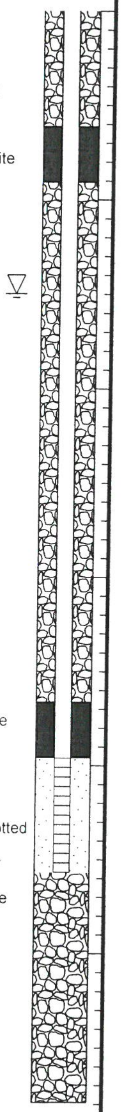
SPT HAMMER: 63.6 kg; drop 0.76 m

BORING METHOD: Power Auger 200 mm Diameter Hollow Stem

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m
0	Ground Surface				
	Stiff to very stiff grey brown to brown CLAYEY SILT, trace sand with occasional sand seams (weathered crust)				
1			1	50 DO	3
2			2	50 DO	2
3			3	50 DO	3
	Firm to stiff grey SILTY CLAY, occasional sand seams	3.50			
5.79	End of borehole	5.79			



PIEZOMETER OR STANDPIPE INSTALLATION



Groundwater level at 1.50 metres below ground surface on March 7, 2007

DEPTH SCALE

1 to 40

Houle Chevrier Engineering Ltd.

LOGGED: PA

CHECKED:

PROJECT: 06-384

LOCATION: Refer to Site Plan, Figure 1

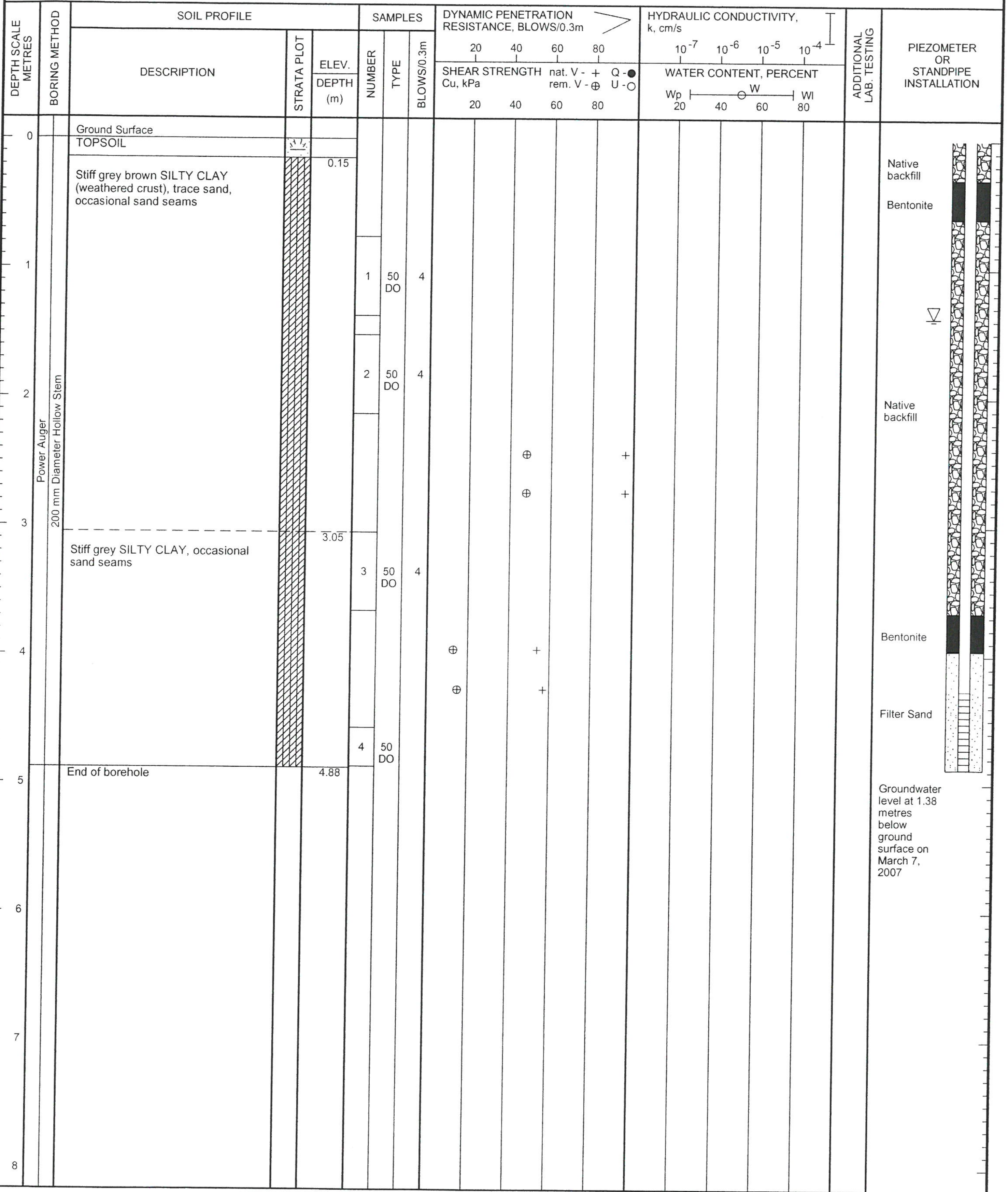
BORING DATE: February 20, 2007

RECORD OF BOREHOLE 3

SHEET 1 OF 1

DATUM:

SPT HAMMER: 63.6 kg; drop 0.76 m



Depth (m)	Description	Number	Type	Blows/0.3m
	Ground Surface			
	TOPSOIL			
0.15	Stiff grey brown SILTY CLAY (weathered crust), trace sand, occasional sand seams	1	50 DO	4
		2	50 DO	4
3.05	Stiff grey SILTY CLAY, occasional sand seams	3	50 DO	4
		4	50 DO	
4.88	End of borehole			

Groundwater level at 1.38 metres below ground surface on March 7, 2007

DEPTH SCALE

1 to 40

Houle Chevrier Engineering Ltd.

LOGGED: JC

CHECKED:

BOREHOLE_RECORD 06-384 BOREHOLE LOGS.GPJ MHECL.GDT 3/28/07

APPENDIX C

Record of Borehole Sheets
Previous Investigation by Houle Chevrier Engineering Ltd.
(Report Number 12-247)

PROJECT: 12-247

RECORD OF BOREHOLE 12-1

SHEET 1 OF 1

LOCATION: See Borehole Location Plan, Figure 2

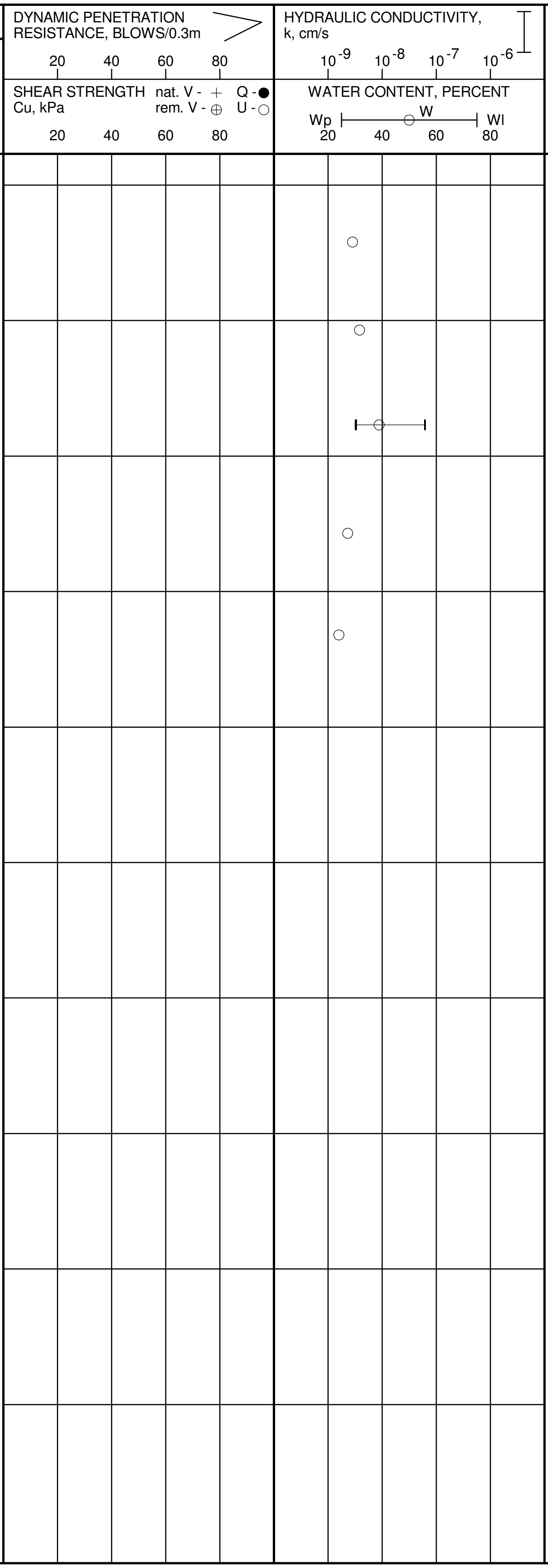
DATUM: Geodetic

BORING DATE: July 4, 2012

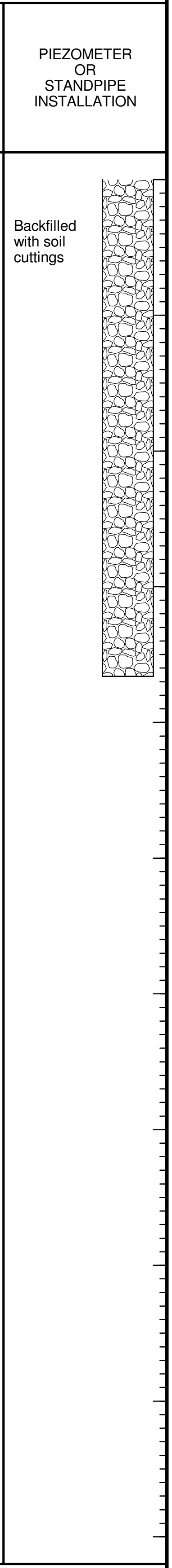
SPT HAMMER: 63.5 kg; drop 0.76 m

BORING METHOD: Power Auger, 200 mm Diameter Hollow Stem

DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m
Ground Surface		105.03			
Grey crushed sand and gravel (FILL MATERIAL)	▨	104.88 0.15			
Grey brown silty clay, trace gravel and organics (FILL MATERIAL)	▨	103.95	1	50 D.O.	15
Former topsoil layer (TOPSOIL)		1.08 103.61	2	50 D.O.	10
Very stiff to stiff, grey brown CLAYEY SILT	▨	1.42 101.37	3 4 5	50 D.O. 50 D.O. 50 D.O.	10 3 3
End of borehole		3.66			



ADDITIONAL LAB. TESTING



DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: M.L.

CHECKED:

PROJECT: 12-247

RECORD OF BOREHOLE 12-2

SHEET 1 OF 1

LOCATION: See Borehole Location Plan, Figure 2

DATUM: Geodetic

BORING DATE: July 4, 2012

SPT HAMMER: 63.5 kg; drop 0.76 m

DEPTH SCALE METRES	BORING METHOD	ROCK PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat. V - + Q - ● rem. V - ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-9} 10^{-8} 10^{-7} 10^{-6} WATER CONTENT, PERCENT Wp ⊢ W ⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m				
0		Ground Surface		103.44							
	Power Auger 200 mm Diameter Hollow Stem	Grey brown silty clay, trace sand and gravel and organic material (FILL MATERIAL)	▩	102.07 1.37	1 2	50 D.O. 50 D.O.	17 13				Backfilled with soil cuttings
		Very stiff to stiff, grey brown SILTY CLAY	▥	101.20 2.24	3	50 D.O.	6				
		Stiff, grey brown CLAYEY SILT	▥	100.09 3.35	4	50 D.O.	4				
		Stiff, grey CLAYEY SILT	▥	99.78 3.66	5	50 D.O.	4				
		End of borehole									

DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: M.L.

CHECKED:

RECORD OF BOREHOLE 12-3

PROJECT: 12-247

LOCATION: See Borehole Location Plan, Figure 2

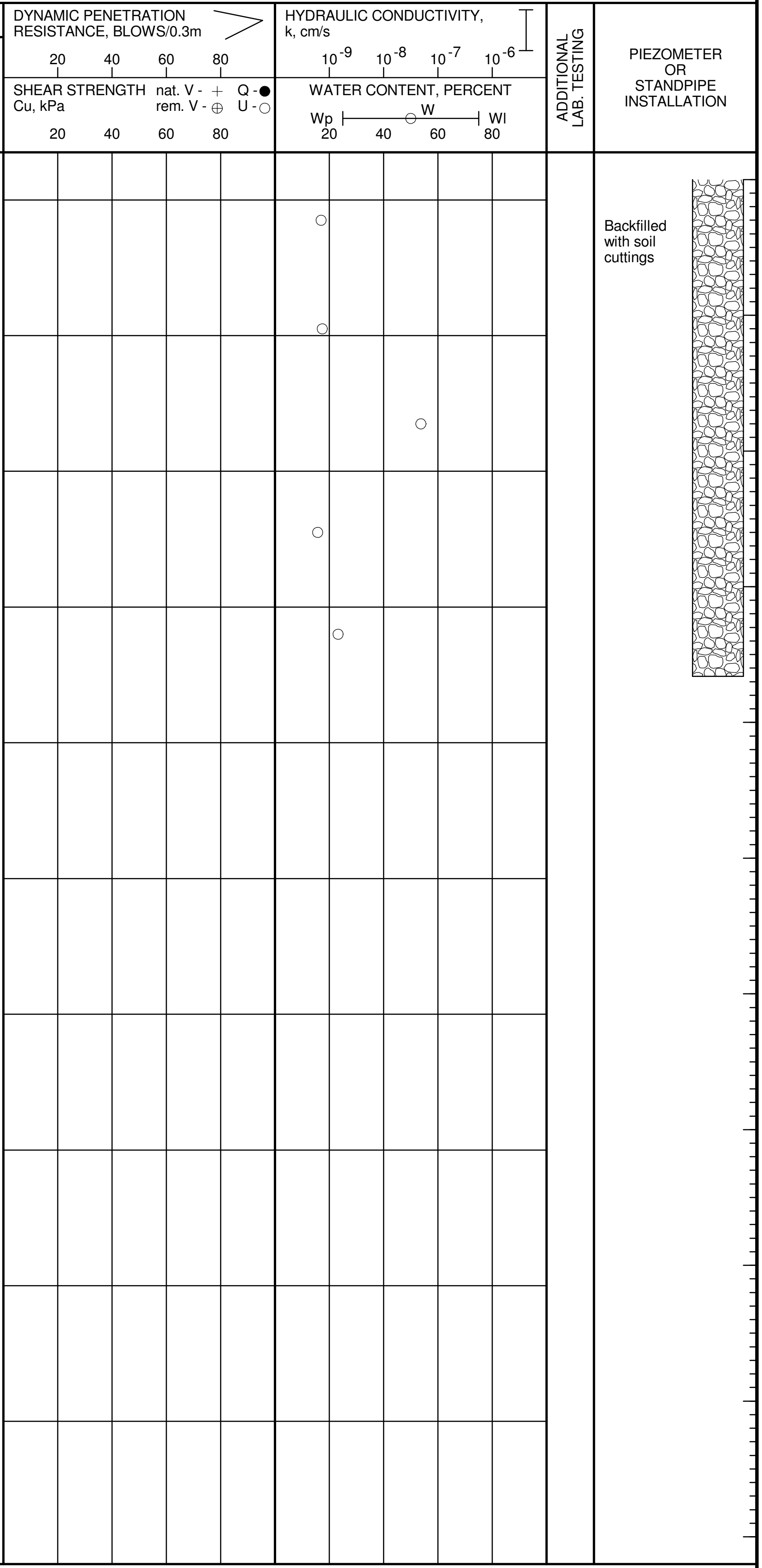
BORING DATE: July 4, 2012

SHEET 1 OF 1

DATUM: Geodetic

SPT HAMMER: 63.5 kg; drop 0.76 m

BORING METHOD	ROCK PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m
Power Auger 200 mm Diameter Hollow Stem	Ground Surface		103.15			
	Grey brown clayey silt, trace gravel and organic material (POSSIBLE FILL)			1	50 D.O.	17
				2	50 D.O.	11
			101.73 1.42			
	Very stiff, grey brown SILTY CLAY			3	50 D.O.	7
			100.56 2.59	4	50 D.O.	3
	Very loose, grey SILT, trace to some clay			5	50 D.O.	3
			99.49 3.66			
	End of borehole					



DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: M.L.

CHECKED:

PROJECT: 12-247

RECORD OF BOREHOLE 12-4

SHEET 1 OF 1

LOCATION: See Borehole Location Plan, Figure 2

DATUM: Geodetic

BORING DATE: July 4, 2012

SPT HAMMER: 63.5 kg; drop 0.76 m

BORING METHOD: Power Auger, 200 mm Diameter Hollow Stem

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat. V - + Q - ● rem. V - ⊕ U - ○ 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-9} 10^{-8} 10^{-7} 10^{-6}

WATER CONTENT, PERCENT: Wp ⊢—○ W —⊣ Wl 20 40 60 80

ADDITIONAL LAB. TESTING

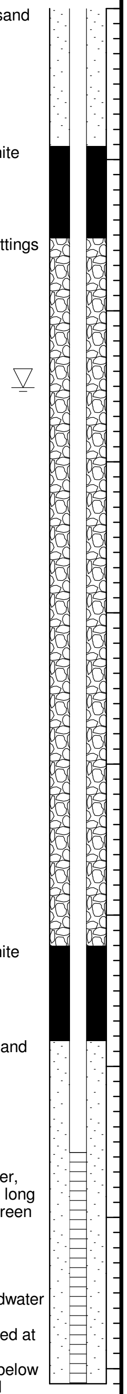
ROCK PROFILE

DESCRIPTION	ELEV. / DEPTH (m)
Ground Surface	102.76
Brown clayey silt, trace organic material (POSSIBLE FILL)	102.15 0.61
Very stiff to stiff, grey brown CLAYEY SILT, some silt layers	100.63 2.13
Compact, grey SILT, trace to some clay	99.71 3.05
Loose to very dense, grey silty sand with gravel, trace clay, possible cobbles and boulders (GLACIAL TILL)	93.66 9.10
Borehole terminated due to practical auger refusal on inferred boulders or bedrock	

SAMPLES

NUMBER	TYPE	BLOWS/0.3m
1	50 D.O.	12
2	50 D.O.	12
3	50 D.O.	10
4	50 D.O.	11
5	50 D.O.	8
6	50 D.O.	3
7	50 D.O.	4
8	50 D.O.	5
9	50 D.O.	36
10	50 D.O.	>50
11	50 D.O.	>50

PIEZOMETER OR STANDPIPE INSTALLATION



Silica sand

Bentonite seal

Soil cuttings

Bentonite seal

Filter sand

51 mm diameter, 1.52 m long well screen

Groundwater level observed at 2.51 m depth below ground surface on July 10, 2012.

DEPTH SCALE 1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: M.L.

CHECKED:

ROCK LOGS 2012 W LAB HC ADJUSTED 12-247 BH LOGS.GPJ HCE DATA TEMPLATE.GDT 21/9/12

PROJECT: 12-247

RECORD OF BOREHOLE 12-5

SHEET 1 OF 1

LOCATION: See Borehole Location Plan, Figure 2

DATUM: Geodetic

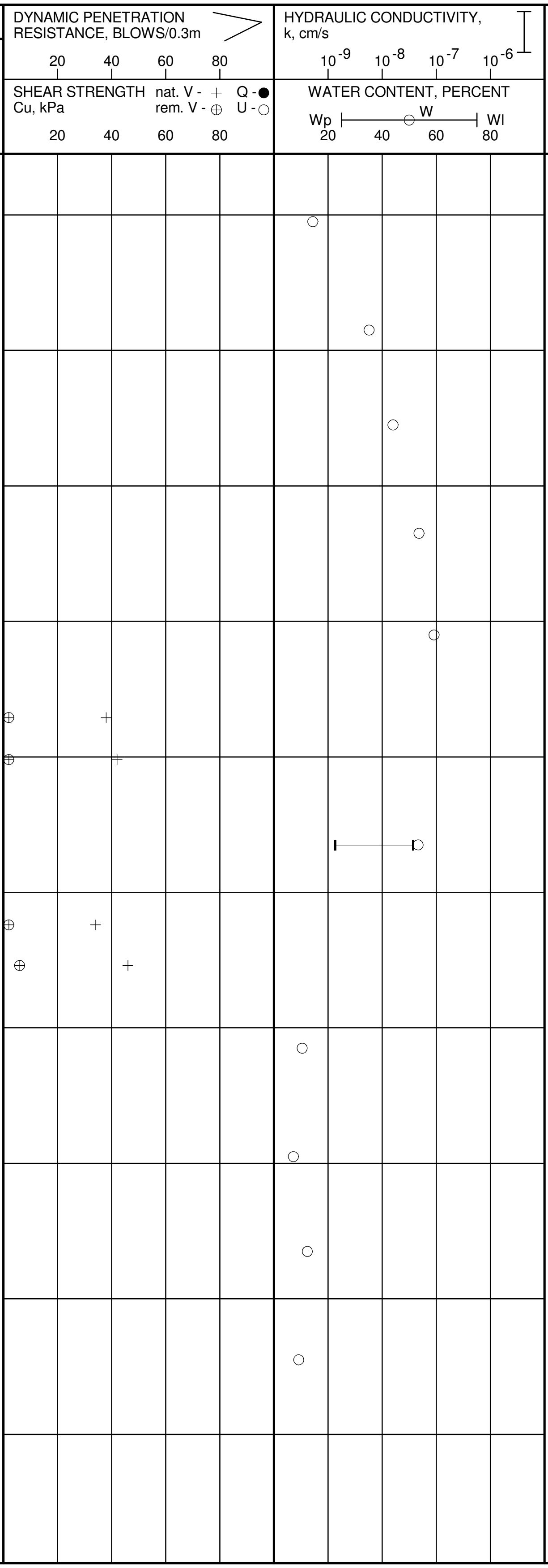
BORING DATE: July 5, 2012

SPT HAMMER: 63.5 kg; drop 0.76 m

BORING METHOD: Power Auger, 200 mm Diameter Hollow Stem

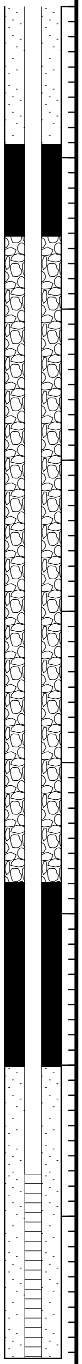
ROCK PROFILE DESCRIPTION	ELEV. DEPTH (m)
Ground Surface	102.25
Brown silty clay, trace organic material (POSSIBLE FILL)	101.64 0.61
Very stiff, grey brown SILTY CLAY	99.25 3.00
Firm, grey SILTY CLAY	96.30 5.95
Loose to very dense, grey silty sand with gravel, trace clay, possible cobbles and boulders (GLACIAL TILL)	93.31 8.94
Borehole terminated due to practical auger refusal on inferred boulders or bedrock	

SAMPLES NUMBER	TYPE	BLOWS/0.3m
1	50 D.O.	12
2	50 D.O.	10
3	50 D.O.	5
4	50 D.O.	2
5	50 D.O.	1
6	50 D.O.	1
7	50 D.O.	9
8	50 D.O.	38
9	50 D.O.	5
10	50 D.O.	>50



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



Groundwater level observed at 1.90 m depth below ground surface on July 10, 2012.

DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: M.L.

CHECKED:

PROJECT: 12-247

RECORD OF BOREHOLE 12-5 B

SHEET 1 OF 1

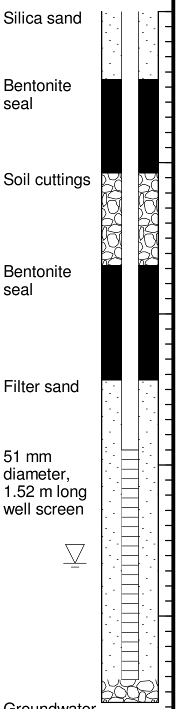
LOCATION: See Borehole Location Plan, Figure 2

DATUM: Geodetic

BORING DATE: July 5, 2012

SPT HAMMER: 63.5 kg; drop 0.76 m

DEPTH SCALE METRES	BORING METHOD	ROCK PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80) SHEAR STRENGTH Cu, kPa nat. V - + Q - ● rem. V - ⊕ U - ○ (20, 40, 60, 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-9}, 10^{-8}, 10^{-7}, 10^{-6}) WATER CONTENT, PERCENT Wp W Wl (20, 40, 60, 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200 mm Diameter Hollow Stem	Ground Surface		102.22							
		Soil conditions not logged									
		(See borehole 12-5 for reference)									
1											
2											
3											
4											
				97.65 4.57							
5											
6											
7											
8											
9											
10											



Groundwater level observed at 3.65 m depth below ground surface on July 10, 2012.

DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: M.L.

CHECKED:

ROCK LOGS 2012 W LAB HC ADJUSTED 12-247 BH LOGS.GPJ HCE DATA TEMPLATE.GDT 21/9/12

APPENDIX D

Record of Borehole Sheets
Previous Investigation by Golder Associates Ltd.
(Report Number 001-2225)

PROJECT: 001-2225

LOCATION: See Site Plan

SAMPLER HAMMER, 64kg; DROP, 760mm

RECORD OF BOREHOLE: 00-1

BORING DATE: Oct. 31, 2000

SHEET 1 OF 1

DATUM: Geodetic

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - O 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—O W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		103.68							
0	Power Auger 200mm Diam (Hollow Stem)	Dark brown silty sand (TOPSOIL)	▨	0.00							
		Grey brown SILTY CLAY (Weathered Crust)	▨	103.38 0.30							
1											
2											
					1	AS	-				
3											
					2	50 DO	4				
		END OF BOREHOLE		100.02 3.66							
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: A.M.C.

CHECKED: TMS

PROJECT: 001-2225

LOCATION: See Site PLan

SAMPLER HAMMER, 64kg; DROP, 760mm

RECORD OF BOREHOLE: 00-2

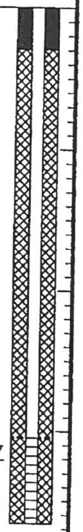
BORING DATE: Oct. 31, 2000

SHEET 1 OF 1

DATUM: Geodetic

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		103.23							
0	Power Auger 200mm Diam (Hollow Stem)	Dark brown silty sand (TOPSOIL)		0.00							Bentonite Seal
		Grey brown becoming grey SILTY CLAY		102.93 0.30							
2					1	AS	-				Native Backfill
3					2	50 DO	2				Standpipe
		END OF BOREHOLE		99.57 3.66							
4											W.L in Standpipe at Elev. 100.02 m Nov. 10, 2000
5											
6											
7											
8											
9											
10											



BOREHOLE 001-2225.GPJ GLDR_CAN.GDT 12 7 00

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: A.M.C.

CHECKED: TMS

PROJECT: 001-2225

RECORD OF BOREHOLE: 00-3

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Oct. 31, 2000

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: STRATA PLOT	SOIL PROFILE: ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		102.39							
0	Power Auger 200mm Diam (Hollow Stem)	Dark brown silty sand (TOPSOIL)		0.00							
		Grey brownSILTY CLAY (Weathered Crust)		0.15							
		Very loose grey sandy silt, some gravel, trace clay (GLACIAL TILL)		99.10 3.29	1	50 DO	2				
		END OF BOREHOLE		98.73 3.66							

Depth scale: 0 1 2 3 4 5 6 7 8 9 10

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: A.M.C.
CHECKED: TMS

PROJECT: 001-2225

RECORD OF BOREHOLE: 00-4

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Oct. 31, 2000

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ├──○W──┤ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m				
		GROUND SURFACE		102.07							
0	Power Auger 200mm Diam (Hollow Stem)	Dark brown silty sand (TOPSOIL)		0.00							
		Grey brown SILTY CLAY (Weathered Crust)	▨	0.08							
1											
2					1	AS	-				
				99.17 2.90							
3		END OF BOREHOLE AUGER REFUSAL									
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: A.M.C.

CHECKED: TMS

PROJECT: 001-2225

RECORD OF BOREHOLE: 00-4A

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Oct. 31, 2000

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: STRATA PLOT	SOIL PROFILE: ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10⁻⁶ 10⁻⁵ 10⁻⁴ 10⁻³); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		GROUND SURFACE		102.07							
	Power Auger 200mm Diam (Hollow Stem)	Probably grey brown Silty Clay (Weathered Crust)	▨	0.00							
1											
2											
3											
4				97.50 4.57							
		END OF BOREHOLE									
5											
6											
7											
8											
9											
10											

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: A.M.C.

CHECKED: TMS

PROJECT: 001-2225

LOCATION: See Site Plan

SAMPLER HAMMER, 64kg; DROP, 760mm

RECORD OF BOREHOLE: 00-4B

BORING DATE: Oct. 31, 2000

SHEET 1 OF 1

DATUM: Geodetic

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: STRATA PLOT	SOIL PROFILE: ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		102.06							
0	Power Auger 200mm Diam (Hollow Stem)	Probably grey brown Silty Clay (Weathered Crust)	▨	0.00							
		END OF BOREHOLE		97.49 4.57							

Depth scale: 0 1 2 3 4 5 6 7 8 9 10

BOREHOLE 001-2225.GPJ GLDR_CAN.GDT 12 5 00

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: A.M.C.

CHECKED: TMS

PROJECT: 001-2225

LOCATION: See Site Plan

SAMPLER HAMMER, 64kg; DROP, 760mm

RECORD OF BOREHOLE: 00-5

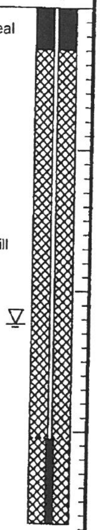
BORING DATE: Oct. 31, 2000

SHEET 1 OF 1

DATUM: Geodetic

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	SOIL PROFILE STRATA PLOT	SOIL PROFILE ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80; SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}; WATER CONTENT PERCENT Wp ├──○W──┤ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		GROUND SURFACE		101.75							
	Power Auger 200mm Diam (Hollow Stem)	Dark brown silty sand (TOPSOIL)		0.00							
		Grey brown becoming grey SILTY CLAY		0.10	1	50 DO	PH				Bentonite Seal Native Backfill Standpipe
		END OF BOREHOLE		98.09 3.66							W.L. in Standpipe at Elev. 99.51 m Nov. 10, 2000



BOREHOLE 001-2225.GPJ GLDR_CAN.GDT 12 7 00

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: A.M.C.

CHECKED: TMS

PROJECT: 001-2225

LOCATION: See Site Plan

SAMPLER HAMMER, 64kg; DROP, 760mm

RECORD OF BOREHOLE: 00-6

BORING DATE: Oct. 31, 2000

SHEET 1 OF 1

DATUM: Geodetic

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ├──○W──┤ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		101.82							
0	Power Auger 200mm Diam (Hollow Stem)	Grey coarse gravel, some sand (FILL)	▩	0.00							
		Brown SILTY fine SAND	░	101.21 0.61							
1											
		Grey brown SILTY CLAY (Weathered Crust)	▨	100.45 1.37							
2					1	AS	-				
3					2	50 DO	2				
		END OF BOREHOLE		98.16 3.66							
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: A.M.C.

CHECKED: TMS

PROJECT: 001-2225

RECORD OF BOREHOLE: 00-6A

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Oct. 31, 2000

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m
		GROUND SURFACE	101.40			
0	Power Auger 200mm Diam (Hollow Stem)	Dark brown silty sand (TOPSOIL)	0.00			
		Grey brown SILTY CLAY (Weathered Crust)	0.08			
1						
2				1	AS	-
3		Grey SILTY CLAY	98.35 3.05			
				2	50 DO	PH
		END OF BOREHOLE	97.74 3.66			
4						
5						
6						
7						
8						
9						
10						

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80

SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: A.M.C.

CHECKED: TMS

PROJECT: 001-2225

RECORD OF BOREHOLE: 00-7

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Oct. 31, 2000

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-8} 10^{-6} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		101.28							
0	Power Auger 200mm Diam (Hollow Stem)	Grey coarse gravel, some sand (FILL)	▩	0.00							
		Grey brown SILTY CLAY (Weathered Crust)	▨	100.58 0.70							
1											
2					1	AS	-				
3					2	50 DO	2				
		END OF BOREHOLE		97.62 3.66							
4											
5											
6											
7											
8											
9											
10											

BOREHOLE 001-2225.GPJ GLDR_CAN.GDT 12 5 00

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: A.M.C.

CHECKED: TMS

APPENDIX E

Record of Borehole Sheets
Previous Investigation by Jacques Whitford Ltd.
(Report Number ONO111725)

Jacques Whitford

BOREHOLE RECORD

1 of **1**

BH04-1

CLIENT Novatech Engineering

LOCATION Hazeldean Road Widening, Iber Rd to Kincardine Dr, Ottawa, ON

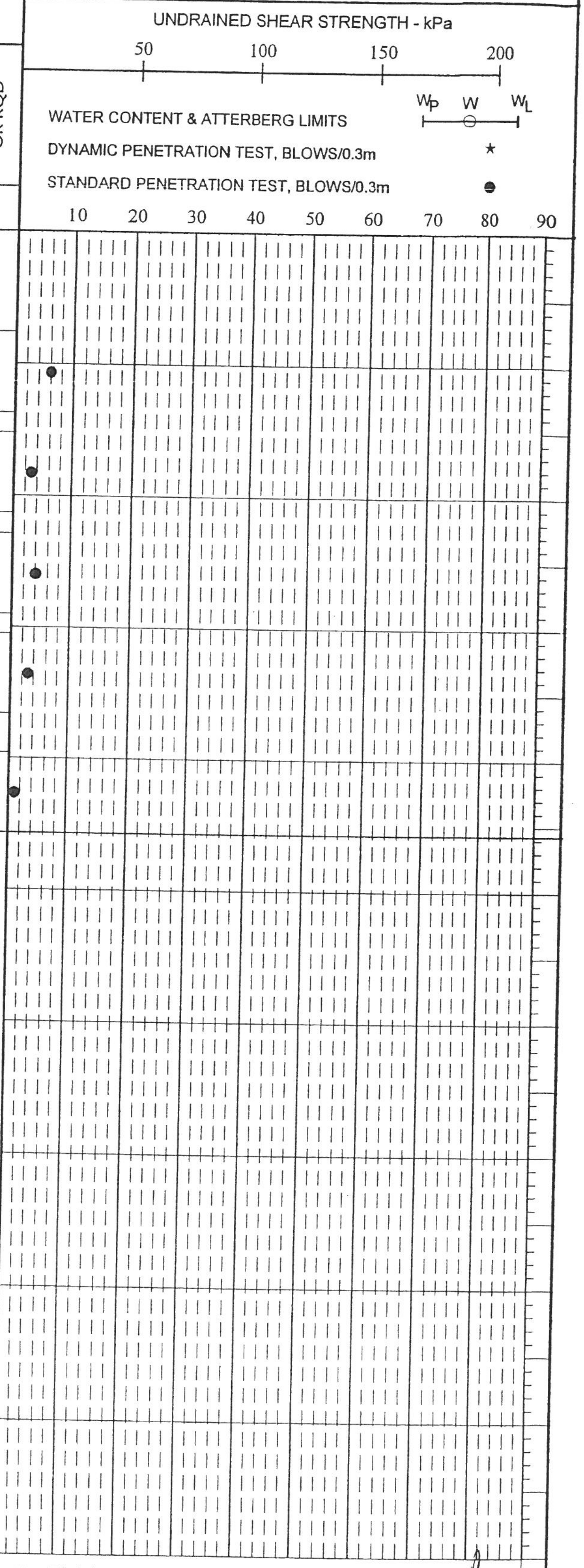
DATES: BORING 08-06-04 WATER LEVEL 25-06-04

BOREHOLE No. BH04-1

PROJECT No. ONO11725

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES			
					TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	**103.67**	**12+702 17.7 Rt**						
		Brown gravelly sand, trace silt: FILL	▨					
1					SS	1	0	6
	102.4	Loose, brownish grey SANDY SILT		▼				
2					SS	2	460	3
					SS	3	610	4
3								
					SS	4	610	3
4	99.4							
	99.1	Soft, grey SILTY CLAY			SS	5	610	1
		End of Borehole Standpipe Installed						
5								
6								
7								
8								
9								
10								



▽ Inferred Groundwater Level

▼

◘ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

App'd

1725.C EL.GD -04

Jacques Whitford

BOREHOLE RECORD

BH04-2

1 of 1

CLIENT Novatech Engineering

LOCATION Hazeldean Road Widening, Iber Rd to Kincardine Dr, Ottawa, ON

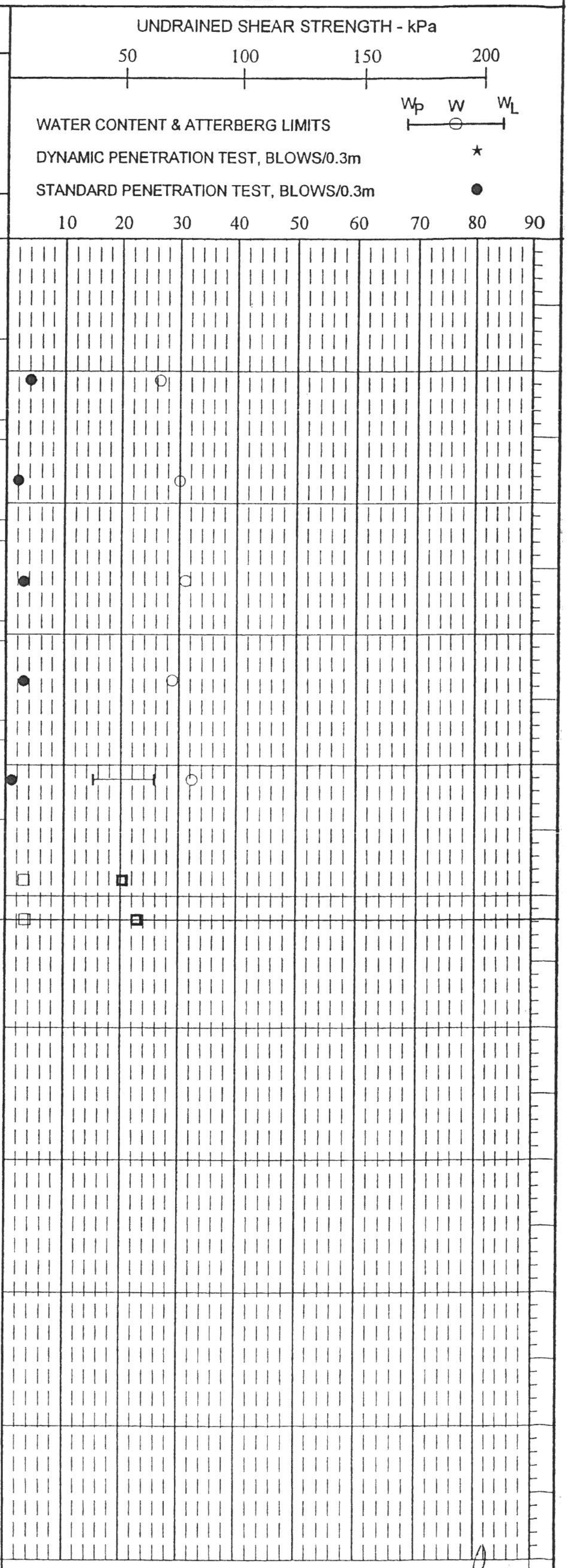
DATES: BORING 04-06-04 WATER LEVEL 25-06-04

BOREHOLE No. BH04-2

PROJECT No. ONO11725

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	**102.92**	**12+700 12.8 Lt**						
	102.8	150 mm TOPSOIL Stiff to firm, brown SILTY CLAY						
1				▼	SS	1	610	4
2					SS	2	610	2
3					SS	3	610	3
					SS	4	610	3
4					SS	5	610	1
5	97.7	End of Borehole Standpipe Installed						
6								
7								
8								
9								
10								



▽ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd

Date 12/7/04

JWL-OLD 11725.GPJ JWEL.GDT 12-07-04

Jacques Whitford

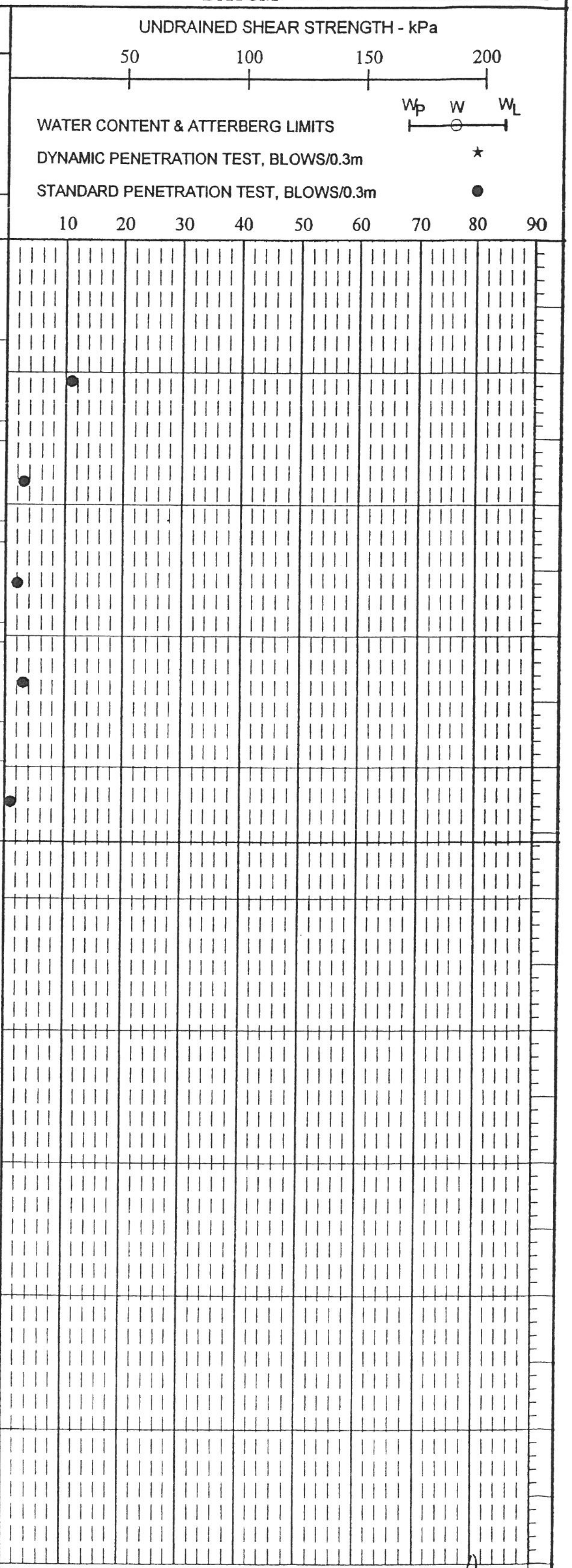
BOREHOLE RECORD

1 of 1

BH04-3

CLIENT Novatech Engineering
LOCATION Hazeldean Road Widening, Iber Rd to Kincardine Dr, Ottawa, ON
DATES: BORING 08-06-04 WATER LEVEL 25-06-04
BOREHOLE No. BH04-3
PROJECT No. ONO11725
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	103.15	12+759 35.88 Rt						
		Brown grey sand, trace silt: FILL						
1					SS	1	0	11
	101.8	Brownish grey fine SANDY SILT		▼				
	101.3	Firm to soft, grey SILTY CLAY						
2					SS	2	610	3
					SS	3	610	2
3					SS	4	610	3
4					SS	5	610	1
	98.6	End of Borehole Standpipe Installed						
5								
6								
7								
8								
9								
10								



▽ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

▫ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date 17/7/04

Jacques Whitford

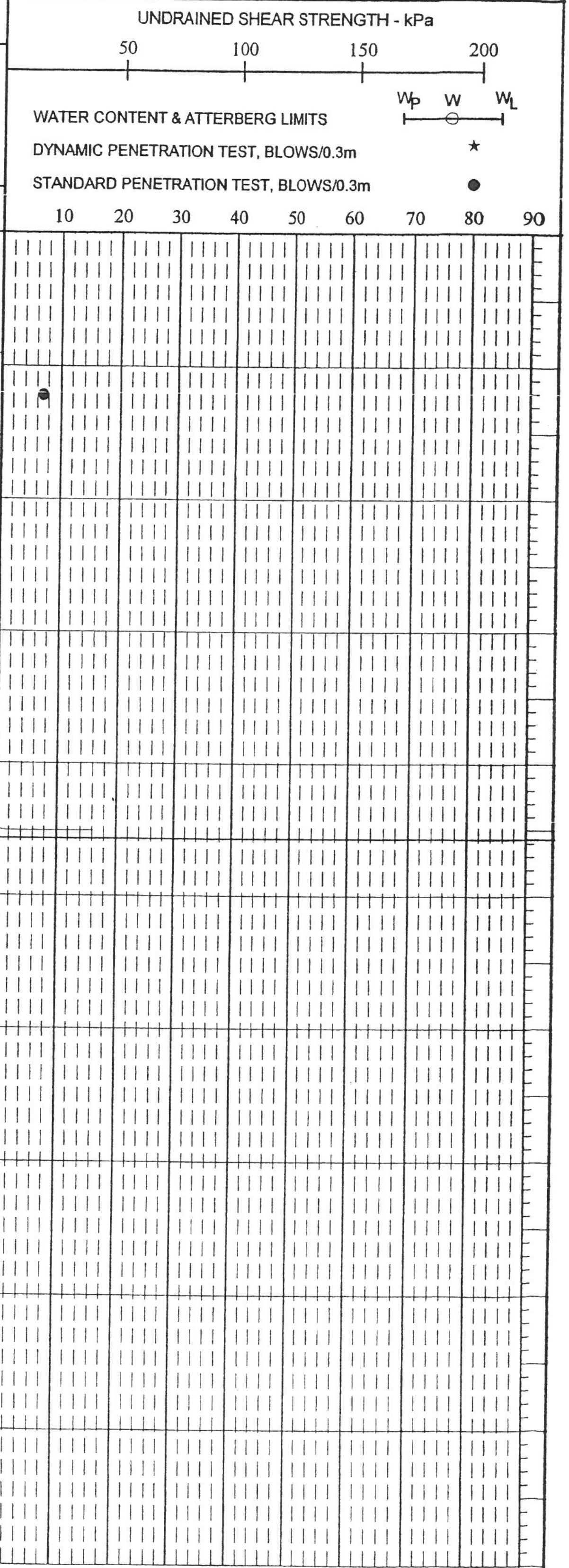
BOREHOLE RECORD

1 of 1

BH04-5

CLIENT Novatech Engineering
LOCATION Hazeldean Road Widening, Iber Rd to Kincardine Dr, Ottawa, ON
DATES: BORING 08-06-04 WATER LEVEL
BOREHOLE No. BH04-5
PROJECT No. ON011725
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	**103.37**	**12+800 1.87 Lt**						
	103.3	100 mm ASPHALTIC CONCRETE						
	103.1	Brown sand and gravel, trace silt: FILL						
	103.0	Brown grey sandy silt, trace gravel: FILL						
1		Brown grey SANDY SILT, trace gravel			SS	1	180	7
2					GS	2		
3				▽				
4					GS	3		
	98.8	End of Borehole						
5								
6								
7								
8								
9								
10								



▽ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

□ Field Vane Test, kPa
□ Remoulded Vane Test, kPa

App'd

Jacques Whitford

BOREHOLE RECORD

BH04-7

1 of 1

CLIENT Novatech Engineering

LOCATION Hazeldean Road Widening, Iber Rd to Kincardine Dr, Ottawa, ON

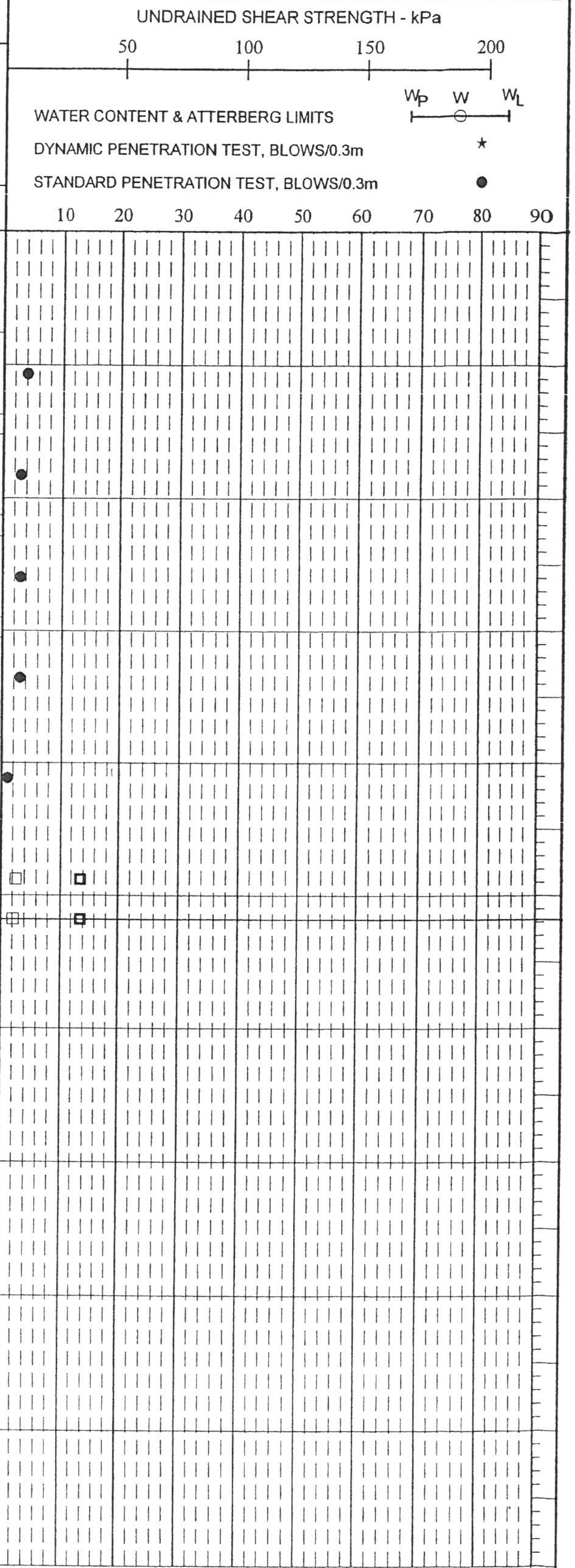
DATES: BORING 04-06-04 WATER LEVEL

BOREHOLE No. BH04-7

PROJECT No. ON011725

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
	101.99	**12+902 12.6 Lt**						
0	101.8	150 mm TOPSOIL						
		Firm to soft, greyish brown SILTY CLAY						
1					SS	1	120	4
2					SS	2	610	3
3					SS	3	610	3
					SS	4	610	3
4					SS	5	610	1
5	96.8	End of Borehole						
6								
7								
8								
9								
10								



∇ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd

Date 12/7/04

Jacques Whitford

BOREHOLE RECORD

BH04-8

1 of 1

CLIENT Novatech Engineering

LOCATION Hazeldean Road Widening, Iber Rd to Kincardine Dr, Ottawa, ON

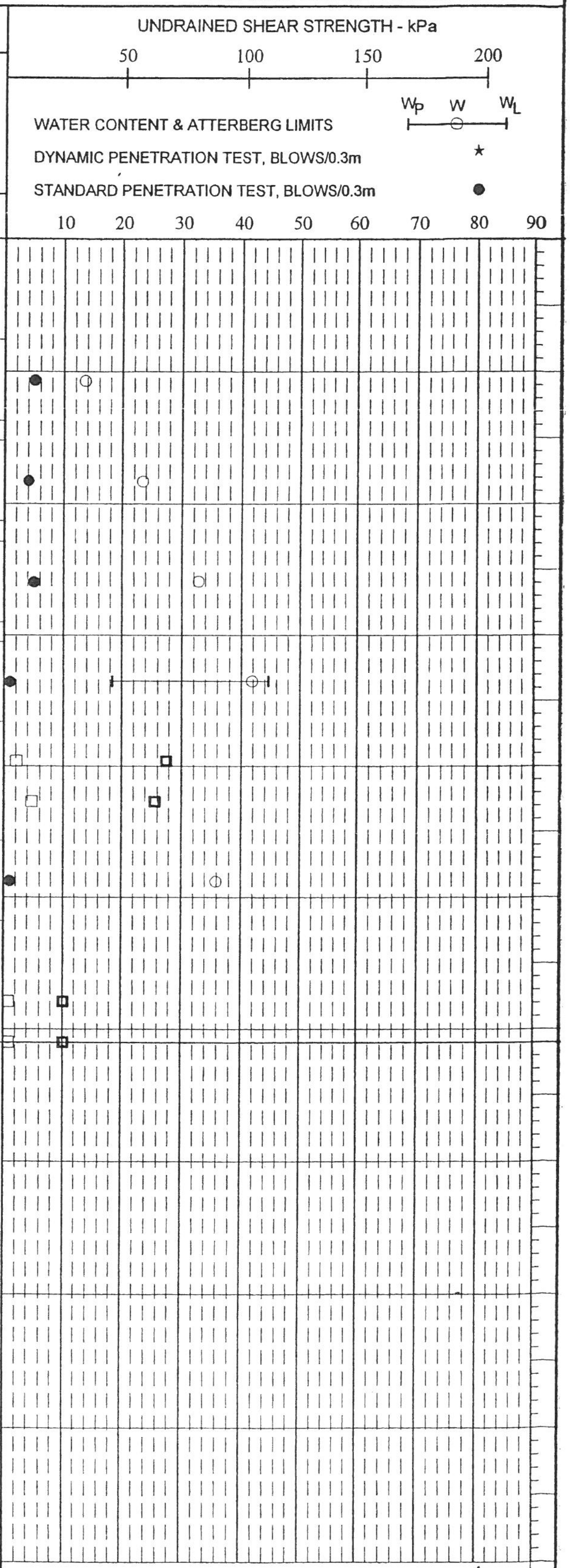
DATES: BORING 07-06-04 WATER LEVEL 25-06-04

BOREHOLE No. BH04-8

PROJECT No. ONO11725

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	101.79	13+017 14.9 Rt						
	101.6	Grey crushed gravel: FILL						
		Dark brown to grey silty clay, some sand and gravel, trace organics: FILL						
1				▼	SS	1	30	5
2					SS	2	200	4
	99.5	Stiff to soft, brown grey SILTY CLAY			SS	3	560	5
3					SS	4	610	1
4								
5					SS	5	610	1
6	95.7	End of Borehole Standpipe Installed						
7								
8								
9								
10								



▽ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

■ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd

Date 12/7/04

Jacques Whitford

1 of 1

BOREHOLE RECORD

BH04-9

CLIENT Novatech Engineering

LOCATION Hazeldean Road Widening, Iber Rd to Kincardine Dr, Ottawa, ON

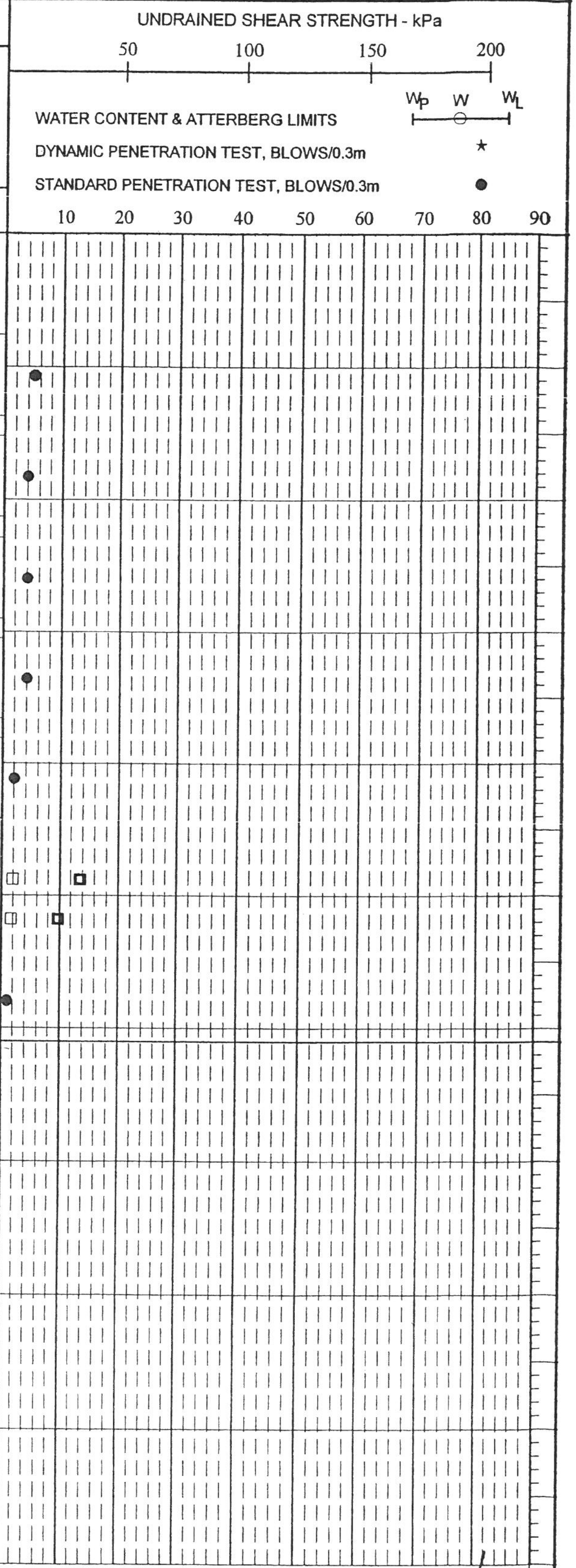
DATES: BORING 03-06-04 WATER LEVEL 03-06-04

BOREHOLE No. BH04-9

PROJECT No. ON011725

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	**101.31**	**13+015 14.1 Lt**						
	101.2	90 mm TOPSOIL						
		Stiff to firm, brown grey SILTY CLAY, some sand seams						
1					SS	1	610	5
2					SS	2	610	4
3					SS	3	610	4
					SS	4	610	4
				∇				
4		- becoming grey and soft			SS	5	610	2
5								
6					SS	6	610	1
	95.2	End of Borehole						
7								
8								
9								
10								



∇ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd

Date 12/7/04

IL-OLD 11725.GPJ JWEL.GDT 12-07-04

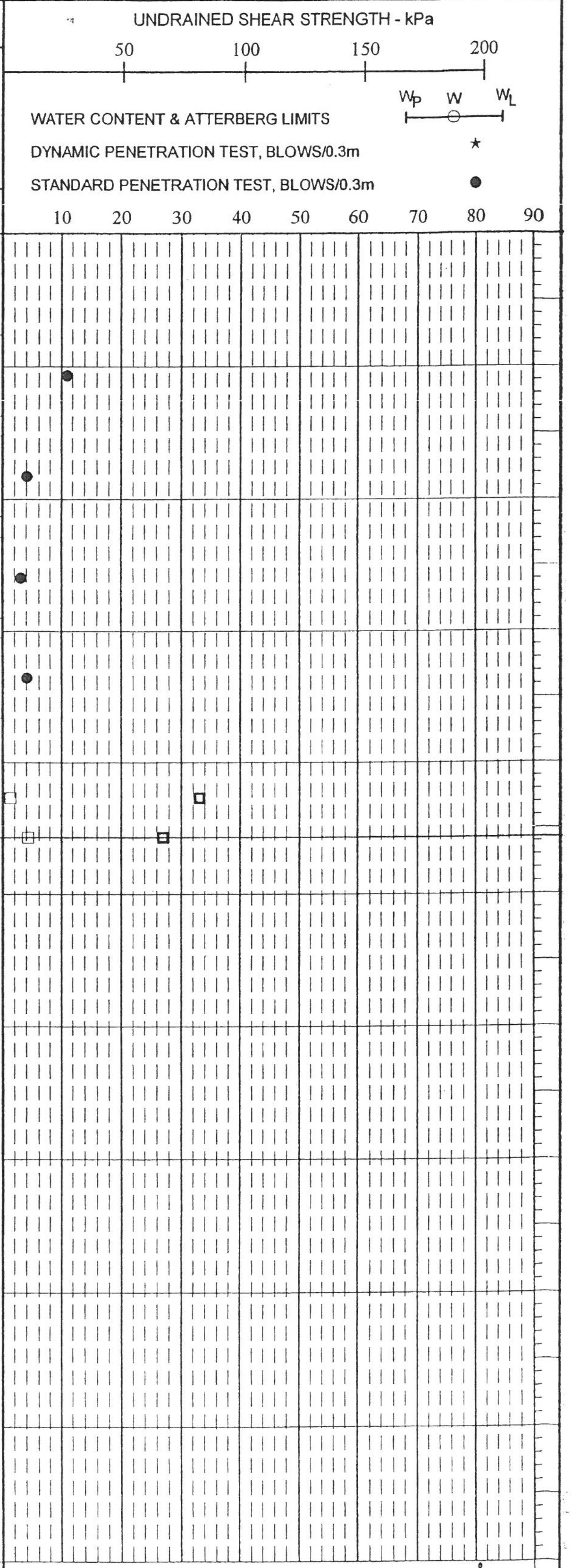
Jacques Whitford

BOREHOLE RECORD

BH04-11

CLIENT Novatech Engineering
LOCATION Hazeldean Road Widening, Iber Rd to Kincardine Dr, Ottawa, ON
DATES: BORING 07-05-04 WATER LEVEL
BOREHOLE No. BH04-11
PROJECT No. ONO11725
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
	101.43	**13+111 12.5 Rt**						
0	101.4	40 mm ASPHALTIC CONCRETE						
	101.2	Brown sand and gravel, trace silt: FILL						
1		Brown silt, some sand: FILL			SS	1	150	11
	100.2							
	99.9	0% Gravel, 17.5% Sand, 82.5 Silt and Clay Compact, brown grey SANDY SILT, trace clay		∇	SS	2	510	4
2		Stiff, brown grey SILTY CLAY, trace sand			SS	3	610	3
3					SS	4	610	4
4	96.9							
		End of Borehole						
5								
6								
7								
8								
9								
10								



∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◻ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date 12/7/04

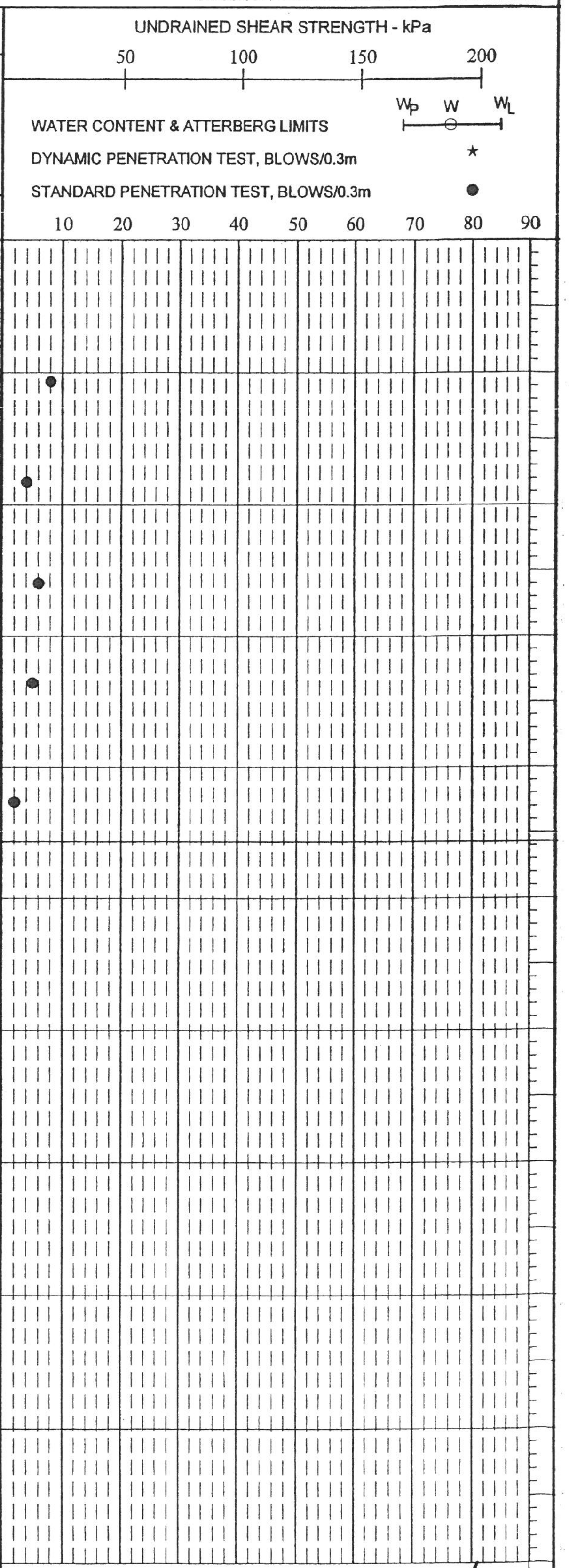
Jacques Whitford

BOREHOLE RECORD

BH04-13

CLIENT Novatech Engineering
LOCATION Hazeldean Road Widening, Iber Rd to Kincardine Dr, Ottawa, ON
DATES: BORING 03-06-04 WATER LEVEL 03-06-04
BOREHOLE No. BH04-13
PROJECT No. ONO11725
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
	99.99	13+213 10.7 Lt						
0	99.9	130 mm TOPSOIL						
		Loose, brown grey fine SANDY SILT, trace clay						
1					SS	1	80	8
2					SS	2	610	4
	97.7	Stiff, brown grey SILTY CLAY, some sand seams			SS	3	610	6
3					SS	4	610	5
4				▽				
		- becoming grey and firm			SS	5	410	2
	95.4	End of Borehole						
5								
6								
7								
8								
9								
10								



▽ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date 12/7/04

Jacques Whitford

BOREHOLE RECORD

BH04-14

1 of 1

CLIENT Novatech Engineering

LOCATION Hazeldean Road Widening, Iber Rd to Kincardine Dr, Ottawa, ON

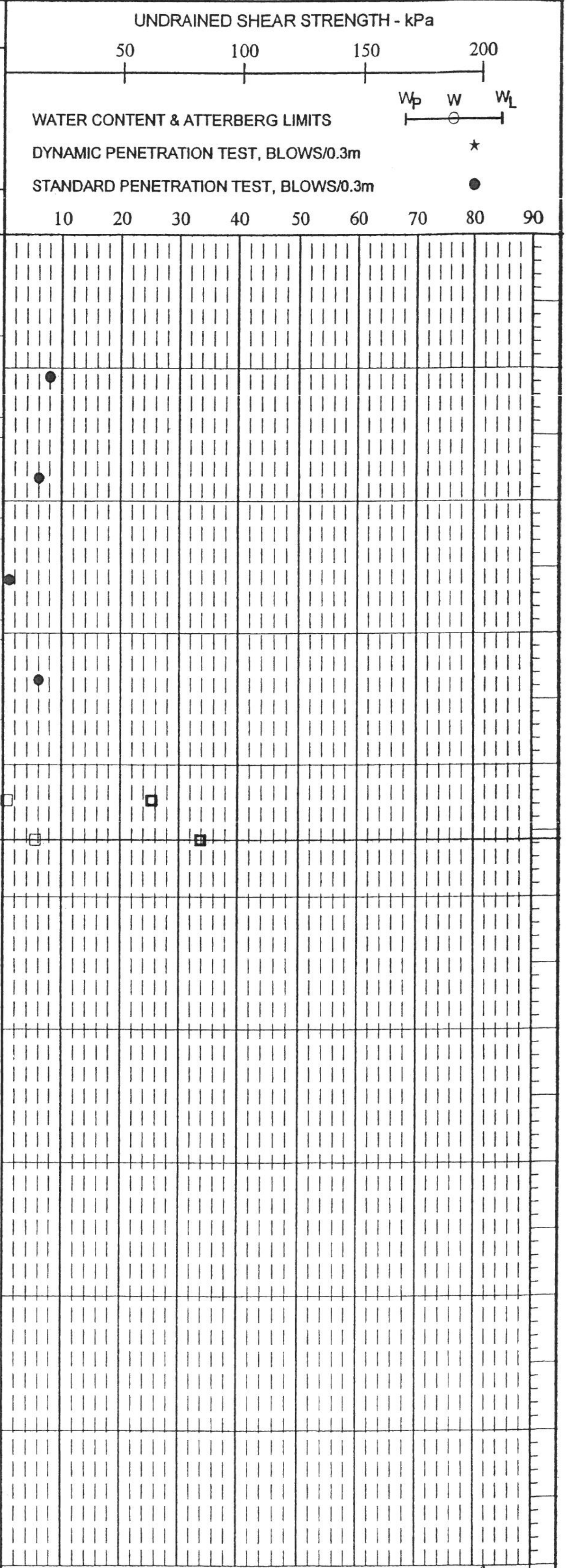
DATES: BORING 06-05-04 WATER LEVEL

BOREHOLE No. BH04-14

PROJECT No. ONO11725

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	**100.58**	**13+311 11.97 Rt**						
	100.5	40 mm ASPHALTIC CONCRETE						
		Compact, brown sand, some gravel, trace silt: FILL						
1					SS	1	0	8
	98.9	Very stiff to stiff, brown grey SILTY CLAY						
2					SS	2	430	6
				∇	SS	3	510	1
3					SS	4	460	6
4								
	96.0	End of Borehole						
5								
6								
7								
8								
9								
10								



∇ Inferred Groundwater Level

□ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

App'd

Jacques Whitford

BOREHOLE RECORD

BH04-17

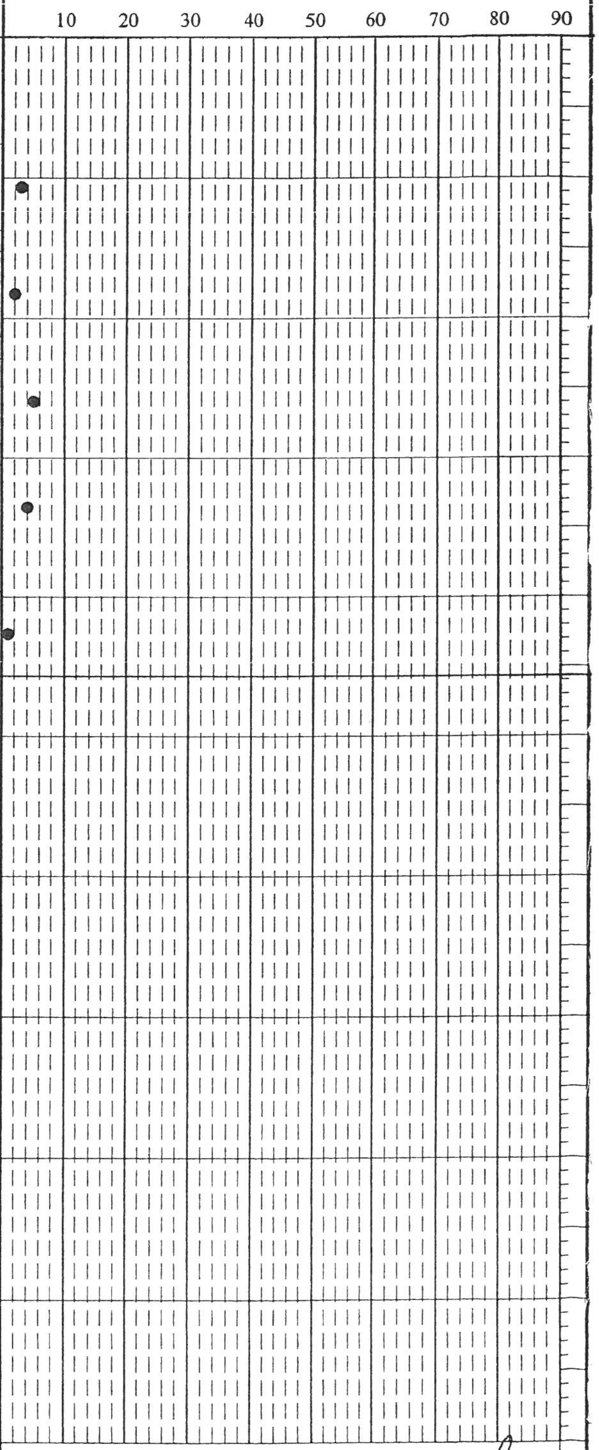
1 of 1

CLIENT Novatech Engineering
LOCATION Hazeldean Road Widening, Iber Rd to Kincardine Dr, Ottawa, ON
DATES: BORING 03-06-04 WATER LEVEL 03-06-04
BOREHOLE No. BH04-17
PROJECT No. ONO11725
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
	99.26	**13+410 9.5 Lt**						
0	99.2	110 mm TOPSOIL						
		Stiff to firm, brown grey SILTY CLAY, trace sand seams						
1					SS	1	80	3
2					SS	2	510	2
					SS	3	510	5
3					SS	4	560	4
				▽				
4		- becoming grey			SS	5	610	1
	94.7	End of Borehole Standpipe Installed						
5								
6								
7								
8								
9								
10								

UNDRAINED SHEAR STRENGTH - kPa
50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●



▽ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

▣ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date 12/7/04

Jacques Whitford

1 of 1

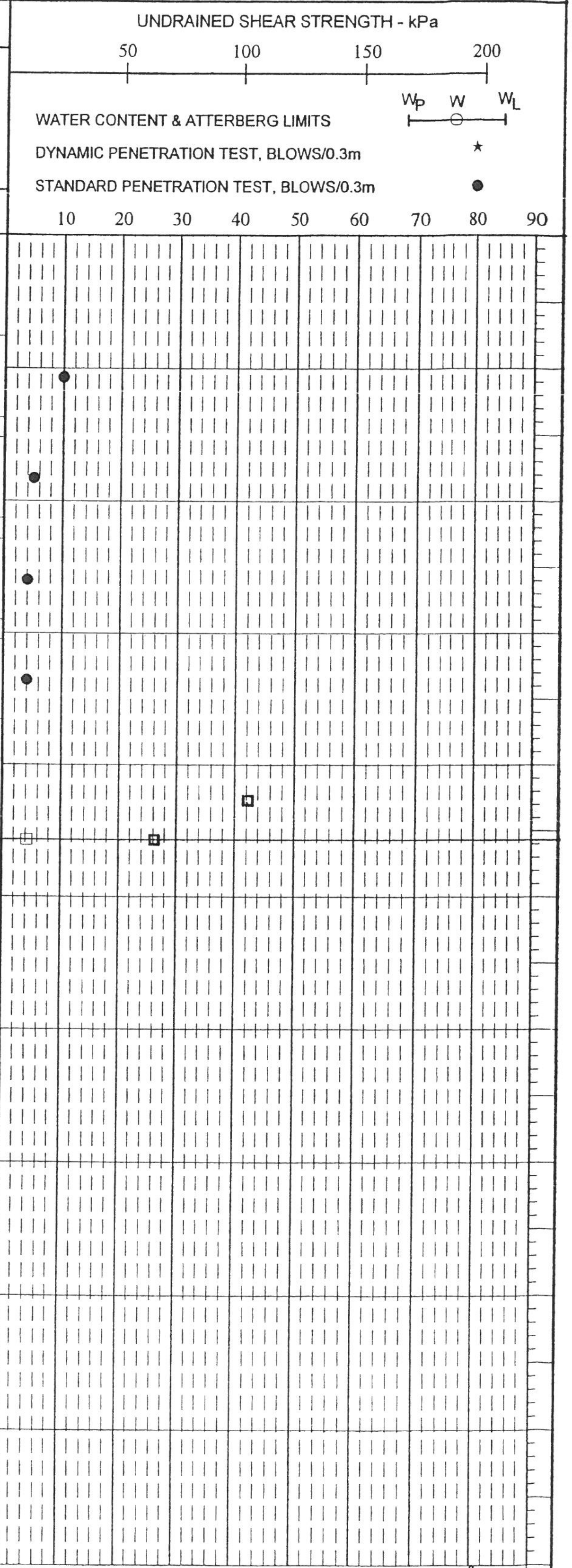
BOREHOLE RECORD

BH04-18

CLIENT Novatech Engineering
LOCATION Hazeldean Road Widening, Iber Rd to Kincardine Dr, Ottawa, ON
DATES: BORING 06-05-04 WATER LEVEL 25-06-04

BOREHOLE No. BH04-18
PROJECT No. ONO11725
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	WATER LEVEL	SAMPLES			
				TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	99.46	**13+508 12.5 Rt**					
	99.4	40 mm ASPHALTIC CONCRETE					
	99.2	Compact, brown sand, some gravel, trace silt: FILL					
1		Compact, brown silt, some sand, trace gravel: FILL		SS	1	0	10
		1.1% Gravel, 18.8% Sand, 80.1% Silt and Clay	▼	SS	2	330	5
2	97.5	Stiff, grey SILTY CLAY		SS	3	580	4
3				SS	4	610	4
4	94.9	End of Borehole Standpipe Installed					



▽ Inferred Groundwater Level

◘ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

App'd

Jacques Whitford

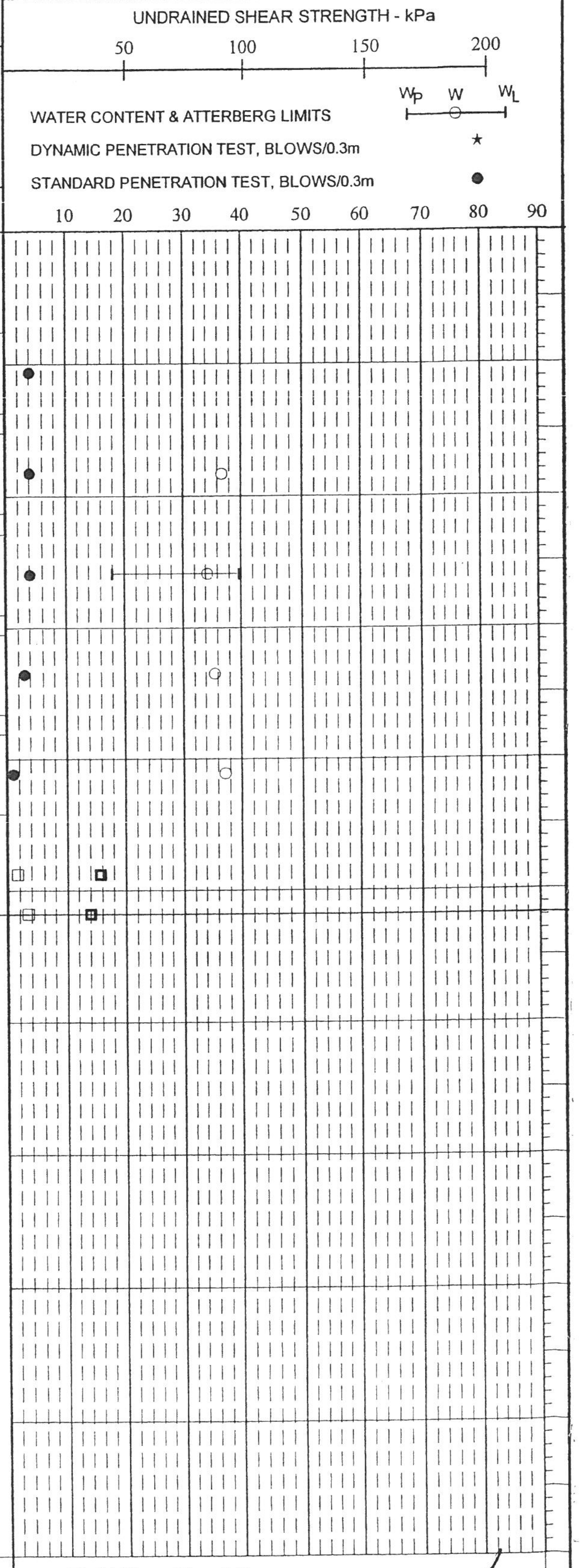
1 of 1

BOREHOLE RECORD

BH04-19

CLIENT Novatech Engineering
BOREHOLE No. BH04-19
LOCATION Hazeldean Road Widening, Iber Rd to Kincardine Dr, Ottawa, ON
PROJECT No. ONO11725
DATES: BORING 04-06-04 WATER LEVEL 25-06-04
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	98.50	**13+509 11.0 Lt**					
	98.4	100 mm crushed gravel: FILL					
	98.3	100 mm ORGANICS					
		Stiff, brown SILTY CLAY					
1				SS	1	0	4
				SS	2	610	4
2				SS	3	610	4
3		- becoming firm and grey	▼	SS	4	610	3
4				SS	5	610	1
5	93.3	End of Borehole Standpipe Installed					
6							
7							
8							
9							
10							



▽ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date 12/7/04

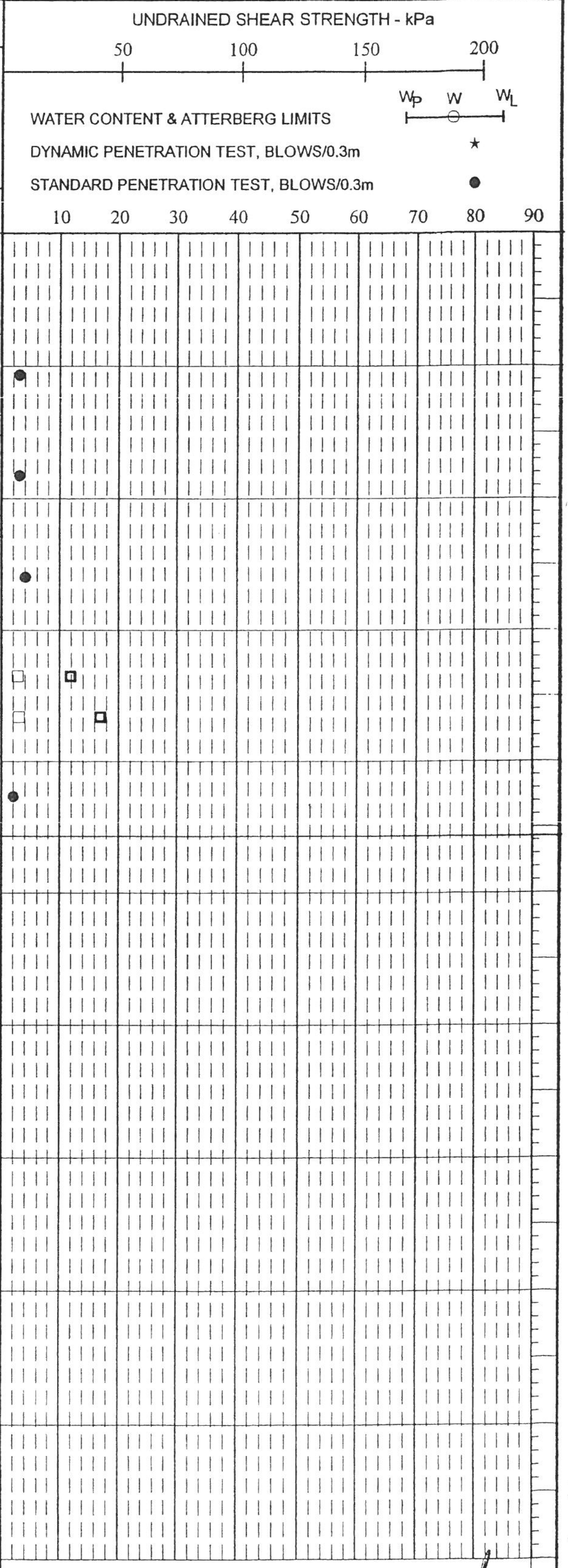
Jacques Whitford

BOREHOLE RECORD

BH04-21

CLIENT Novatech Engineering
LOCATION Hazeldean Road Widening, Iber Rd to Kincardine Dr, Ottawa, ON
DATES: BORING 03-06-04 WATER LEVEL
BOREHOLE No. BH04-21
PROJECT No. ONO11725
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	**97.87**	**13+609 2.0 Lt**						
	97.8	80 mm TOPSOIL						
		Very loose, brown grey fine SANDY SILT, trace clay						
1					SS	1	0	3
2					SS	2	360	3
	95.5	Firm, grey SILTY CLAY, some sand seams			SS	3	560	4
3								
4					SS	4	600	2
	93.3	End of Borehole						
5								
6								
7								
8								
9								
10								



∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

◘ Field Vane Test, kPa
☐ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

App'd
Date 12/7/04

geotechnical

environmental

hydrogeology

materials testing & inspection

experience • knowledge • reliability